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Proceedings of the American Society of Civil Engineers

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June, 1960

Papers

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

JET DIFFUSION IN LIQUID OF GREATER DENSITY

By G. Abraham[1]

SYNOPSIS

A study on the influence of the density difference between a jet and the surrounding fluid on the diffusion by a circular vertical submerged turbulent jet is presented. The results are summarized in Figs. 1(a) and 1(b), which show how the density and the velocity along the axis of the jet depend on the initial density difference between the jet and the surrounding fluid and on the velocity of the jet when it issues from a nozzle. The theoretical basis for the families of curves of Fig. 1 is described together with laboratory experiments, which confirmed the theoretical considerations. The experiments were performed with water jets. The density of the jet was 1,000 kg per cu m (fresh water), the density of the surrounding fluid (salt solution) varied from 1,020 to 1,050 kg per cu m. Because of the range of densities covered by the experiments the results apply to the case of sewage disposal in the marine environment.

INTRODUCTION

For the case of a turbulent jet surrounded by homogeneous still water, the degree of dilution depends on the quantity and velocity of flow, the diameter of the outlet-pipe, and the density difference between the fluid of the jet and the surrounding water. The relationship between these factors is described here for the case of a turbulent jet issuing vertically upward in surrounding water with a specific weight greater than that of the jet. Theoretical information is

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[1] Hydr. Engrg. Lab., Wave Research Projects, Univ. of California, Berkeley, Calif.

presented together with results of laboratory tests. The range of density differences covered by the experiments is such that the results obtained apply to the case of sewage disposal in the marine environment.

THEORY

In the case of homogeneous surrounding water with no currents, the velocity u and the concentration c at any point of the jet depend in general on the following variables:

x = the distance from the nozzle of the outlet-pipe measured along the axis of the jet;

y = the distance from the axis of the jet;

d = the diameter of the outlet-pipe;

u_o = the velocity of the fluid of the jet at $x = 0$;

ρ_o = the density of the fluid of the jet at $x = 0$;

$\Delta\rho$ = $\rho_s - \rho_o$: the density difference between the fluid of the jet at $x = 0$ and the surrounding fluid; and

ν = the kinematic viscosity of the fluid of the jet at $x = 0$.

The concentration c is defined as:

$$c = \frac{\rho - \rho_s}{\rho_o - \rho_s} \quad \text{.................... (1)}$$

in which ρ is the density at any point of the jet and ρ_s the density of the surrounding fluid.

Dimensional analysis shows[2] that the velocity u and the concentration c depend on the other variables as:

$$\frac{u}{u_o} = f\left(\frac{x}{d}, \frac{y}{d}, \frac{\Delta\rho}{\rho_o}, F, R\right) \quad \text{.............. (2)}$$

and

$$\frac{c}{c_o} = f\left(\frac{x}{d}, \frac{y}{d}, \frac{\Delta\rho}{\rho_o}, F, R\right) \quad \text{.............. (3)}$$

in which F is a Froude number,

$$F = \frac{u_o}{\sqrt{(\Delta\rho/\rho_o)\, g\, d}} \quad \text{.................. (4)}$$

and R is a Reynolds number,

$$R = \frac{u_o\, d}{\nu} \quad \text{.................... (5)}$$

The Reynolds number gives the ratio of the inertia forces to the viscous forces. Since this paper is restricted to turbulent jets, the Reynolds number is considered to be sufficiently large to have no influence on the functions of Eqs. 2 and 3.

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2 "Diffusers for Disposal of Sewage in Sea Water," by A M Rawn, F. R. Bowerman and N. H. Brooks, Proceedings, ASCE, Vol. 86, No. SA 2, March, 1960.

The Froude number gives the ratio of the inertia forces to the gravitational forces. If at any point of the jet the local Froude number based on the local velocity and the local density-difference with the surrounding water is large, the influence of the gravitational forces may be neglected at that point. It is evident that the inertia forces are less important at greater distances from the nozzle. Hence it is possible that near the nozzle, the Froude number is sufficiently large to neglect the influence of the buoyancy while it may be that at greater distances from the nozzle the buoyancy has to be considered.

According to F. H. Schmidt[3] the problem may be divided into the following parts:

1. The non-buoyancy case, defined by $F = \infty$. For this case the buoyancy is negligible at any point of the jet.
2. The buoyancy case, defined by $F \to 0$. For this case the initial momentum of the jet is negligible.
3. The intermediate case, defined by a finite value of F. For this case the initial momentum is predominant near the nozzle, the buoyancy at larger distances from the nozzle.

A discussion of the various parts of the problem follows.

The Non-Buoyancy Case, $F = \infty$.—Studies regarding the non-buoyancy case were conducted by M. L. Albertson, Y. B. Dai, R. A. Jensen and H. Rouse,[4] W. Forstall and E. W. Gaylord,[5] J. O. Hinze and B. G. van der Hegge Zynen,[6] S. Corrsin and M. S. Uberoi,[7] and W. R. Keagy; A. E. Weller, F. A. Reed and W. T. Reid.[8]

These studies yield the following results:

For the zone of flow establishment ($\frac{x}{d} < 6$ or 7)

$$\frac{u}{u_o} = e - k\left(y + C_3\,x - \frac{d}{2}\right)^2 \Big/ x^2 \qquad \text{(6a)}$$

for $y > \frac{d}{2} - C_3\,x$,

$$\frac{u}{u_o} = 1 \qquad \text{(6b)}$$

for $y < \frac{d}{2} - C_3\,x$;

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3 "On the Diffusion of Heated Jets," by F. H. Schmidt, Sartoyck ur Tellus, Vol. 9, No. 3, August, 1957, pp. 378-383.

4 "Diffusion of Submerged Jets," by M. L. Albertson, Y. B. Dai, R. A. Jensen and H. Rouse, Transactions, ASCE, Vol. 115, 1950, pp. 639-664.

5 "Momentum and Mass Transfer in a Submerged Water Jet," by W. Forstall and E. W. Gaylord, Journal of Applied Mechanics, Vol. 22, No. 2, June, 1955, pp. 161-164.

6 "Transfer of Heat and Matter in the Turbulent Mixing Zone of an Axially Symmetrical Jet," by J. O. Hinze and B. G. van der Hegge Zynen, Seventh Internatl. Congress for Applied Mechanics, London, 1948, pp. 286-299.

7 "Further Experiments on the Flow and Heat Transfer in a Heated Turbulent Air Jet," by S. Corrsin and M. S. Uberoi, N.A.C.A. Tech. Note N 1865, April, 1949.

8 "Mixing in Inhomogeneous Gas Jets," by W. R. Keagy, A. E. Weller, F. A. Reed and W. T. Reid, Batelle Memorial Inst., The Rand Corp., Santa Monica, Calif., February, 1949.

$$\frac{c}{c_o} = e^{-\mu k\left(y + C_4 x - \frac{d}{2}\right)^2 x^2} \qquad (7a)$$

for $y > \frac{d}{2} - C_4 x$,

$$\frac{c}{c_o} = 1 \qquad (7b)$$

for $y < \frac{d}{2} - C_4 x$.

For the zone of established flow $\left(\frac{x}{d} > 6 \text{ or } 7\right)$

$$\frac{u}{u_m} = e^{-k(y/x)^2} \qquad (8)$$

and

$$\frac{u_m}{u_o} = C_1 \frac{d}{x} \qquad (9)$$

for $\frac{x}{d} > C_1$;

$$\frac{c}{c_m} = e^{-\mu k (y/x)^2} \qquad (10)$$

and

$$\frac{c_m}{c_o} = C_2 \frac{d}{x} \qquad (11)$$

for $\frac{x}{d} > C_2$;

in which u_m is the velocity at the axis of the jet $(y = 0)$, c_m is the concentration at the axis of the jet, and μ, k, C_1, C_2, C_3 and C_4 are dimensionless constants. To obtain a smooth transition between the zones it is necessary that

$$C_3 = \frac{1}{2} \frac{1}{C_1} \qquad (12)$$

and

$$C_4 = \frac{1}{2} \frac{1}{C_2} \qquad (13)$$

The experimental values for the constants μ, k, C_1 and C_2, and the corresponding values of C_3 and C_4 are shown in Table 1.

The experiments of Corrsin and Uberoi[7] show that in the range $1 < \frac{\rho_s}{\rho_o} < 2$ (the range covered by their experiments), $k = 96\left[1 + 0.19\left(\frac{\rho_s}{\rho_o} - 1\right)\right]^{-2}$ and $\mu = 0.7$ = constant. As a consequence the constants k and μ do not vary much for $1 < \frac{\rho_s}{\rho_o} < 1.025$, that is, within the range of $\frac{\rho_s}{\rho_o}$ values that are of interest in sewage disposal problems in the ocean.

Hence for $1 < \frac{\rho_s}{\rho_o} < 1.025$ we may describe the experimental results by average values for k and μ and according to theory by average values for C_1 and C_2. Since water jets are involved in the disposal of sewage, the following have been selected: $k = 77$, $\mu = 0.8$, $C_1 = 6.4$, $C_2 = 5.2$, $C_3 = 0.078$, and $C_4 = 0.096$. These are the values obtained in the experiments of Forstall and Gaylord.[5]

TABLE 1.—PREVIOUSLY PUBLISHED DATA FOR NON-BUOYANCY CASE

Author	Type of jet	$\frac{\rho_s}{\rho_o}$	Experimental values				Calculated values	
			k	μ	C_1	C_2	C_3 (Eq. 12)	C_4 (Eq. 13)
Albertson et al.	air jets	exactly 1	76	...	6.2	...	0.080	...
Forstall and Gaylord	water jets salt solutions	1.01	77	0.8	6.4	5.2	0.078	0.096
Keagy et al.	mixture of gasses	1.03	88	0.71	5.8	5.9	0.086	0.085
Hinze and van der Hegge Zynen	mixture of gasses	1.01	100	0.74	...	...	...	...
Corrsin and Uberoi	hot air jets	about 1	96	0.70	6.6	5.4	0.076	0.092

The Buoyancy Case ($F \rightarrow 0$).—Studies on the buoyancy case were made by B. R. Morton, G. Taylor and J. S. Turner[9] and by Rouse, C. S. Yih and H. W. Humphreys.[10]

The theoretical work of Morton, et al.,[9] yielded the following results:

$$u_m = \frac{5}{6} \alpha \left(\frac{9}{10} \alpha Q\right)^{1/3} x^{-1/3} \qquad (14)$$

and

$$g \frac{(\rho_s - \rho_m)}{\rho_s} = \frac{5}{6} \frac{Q}{\alpha} \left(\frac{9}{10} \alpha Q\right)^{-1/3} x^{-5/3} \qquad (15)$$

in which α is a dimensionless constant and $Q = \frac{\pi}{4} d^2 u_o \frac{\rho_s - \rho_o}{\rho_o} g$.

These formulas have been derived assuming that the velocity and buoyancy force are constant across the jet and zero outside of it and assuming that the jet had no initial momentum.

9 "Turbulent Gravitational Convection from Maintained and Instantaneous Sources," by B. R. Morton, G. Taylor and J. S. Turner, Proceedings, Royal Soc. of London, Series A, March 6, 1956, 11, pp. 1-23.

10 "Gravitational Convection from a Boundary Source," by H. Rouse, C. S. Yih and H. W. Humphreys, Sartoyck ur Tellus, Vol. 4, No. 3, August, 1952, pp. 201-210.

The formulas are only valid for a jet satisfying d = 0. For a jet with finite diameter the factor x of Eqs. 14 and 15 must be replaced by the factor (x + 2d).

The constant α must be found by experiment. Experiments made with water jets indicated that $\alpha = 0.093$.

It is possible to write the equivalent of Eqs. 14 and 15 for a jet with a finite diameter in a dimensionless form by use of the relationships in Eqs. 4 and $\alpha = 0.093$. This gives

$$\frac{u_m}{u_o} = 3.65\ F^{-2/3}\left(\frac{x}{d} + 2\right)^{-1/3} \qquad (16)$$

and

$$\frac{c_m}{c_o} = 9.7\ F^{2/3}\left(\frac{x}{d} + 2\right)^{-5/3} \qquad (17)$$

The experiments of Morton, et al.,[9] showed that the distribution perpendicular to the axis of the jet is given by the Gaussian curves:

$$u = u_m\ e^{-80(y/x)^2} \qquad (18)$$

and

$$c = c_m\ e^{-80(y/x)^2} \qquad (19)$$

Rouse, et al.,[10] also derived the 5/3 and 1/3 power of Eqs. 16 and 17 theoretically. Their semi-empirical formulas, based on experiments done with air jets reduce in dimensionless form to:

$$\frac{u_m}{u_o} = 4.35\ F^{-2/3}\left(\frac{x}{d}\right)^{-1/3} \qquad (20)$$

$$\frac{c_m}{c_o} = 9.35\ F^{2/3}\left(\frac{x}{d}\right)^{-5/3} \qquad (21)$$

$$u = u_m\ e^{-96(y/x)^2} \qquad (22)$$

and

$$c = c_m\ e^{-71(y/x)^2} \qquad (23)$$

The author's experimental results agree better with Eqs. 16 and 17, due to Morton, et al., than with Eqs. 20 and 21, due to Rouse, et al., since the first set of equations takes a small initial momentum in account. Therefore the author has preferred to apply Eqs. 16 and 17 in the following section.

The Intermediate Case, Finite Values of F.—The intermediate case has been studied theoretically by Schmidt.[3] The equations that he derived suggest that the intermediate case may be described approximately by the equations for the non-buoyancy case (Eqs. 6, 7, 8, 9, 10, 11 and the previously selected values

of k, μ, C_1, C_2, C_3 and C_4) from the nozzle to a certain distance from the nozzle, and by the equations for the buoyancy case (Eqs. 16 and 17) at greater distances from the nozzle. The transition occurs at the point $x = x_t$, where the curves that describe the non-buoyancy case and the buoyancy case intersect each other (Fig. 1).

For sufficiently large values of F (F > about 10) the ordinates x_t of the two sets of points of intersection satisfy

$$\frac{2\ C_2}{\mu\ k}\ \frac{1}{F^2}\left(\frac{x_t}{d}\right)^2 \approx 1 \quad \ldots\ldots\ldots\ldots\ldots (24)$$

with C_2, μ and k according to the previously selected values.

For sufficiently large values of x_t and hence for sufficiently large values of F

$$\frac{2\ C_2}{\mu\ k}\ \frac{1}{F^2}\left(\frac{x_t}{d}\right)^2 = \frac{g \int_o^{x_t} dx \int_o^{\infty} 2\ \pi\ y\ \left(\rho_s - \rho\right) dy}{\frac{\pi\ d^2}{4}\ \rho_o\ u_o^{\ 2}} \quad \ldots\ldots (25)$$

since for sufficiently large values of x_t the contribution of the zone of flow establishment to the integral of Eq. 25 is relatively small and then it does not make much difference if we apply Eqs. 10 and 11 also for $\frac{x}{d} < 6$ or 7 to evaluate the integral.

Eqs. 24 and 25 show, for sufficiently large values of F at least, that the influence of the initial momentum is larger than the influence of the buoyancy force for $x < x_t$, while the contrary applies for $x > x_t$. This is compatible with the description of the intermediate case suggested by the equations of Schmidt of which Fig. 1 is a graphical representation. In the following section experiments will be described by which the curves of Fig. 1 are confirmed.

Fig. 1 only applies for the case that the initial momentum and the buoyancy force have the same direction. For the case that the initial momentum and the buoyancy force have opposite direction, the distance x_t is an approximation of the distance from the nozzle, where the velocity along the axis will be reduced to zero.

EXPERIMENTS

The experiments to check the families of curves of Fig. 1 were made in a glass-wall tank, with a height of 3 ft and a cross section of 1 ft by 1 ft. The jet issued into the tank through pipes through the tank bottom with internal diameters of 0.6 in., 0.8 in. and 1.0 in. The end of the vertical pipes was about 4 in. above the bottom of the tank. The upper edge of the tank was constructed as a continuous weir. The discharge of the jet was determined by a volumetric measurement of the discharge over the weir.

The density of the fluid of the jet was about 1,000 kg per cu m. The density of the surrounding fluid varied from 1,020 kg per cu m to 1,050 kg per cu m. The surrounding fluid was a solution of crude table salt and water.

Measurements of the Density.—The density at the axis of the jet was measured to check the curves for the concentration. Samples of the fluid of the jet

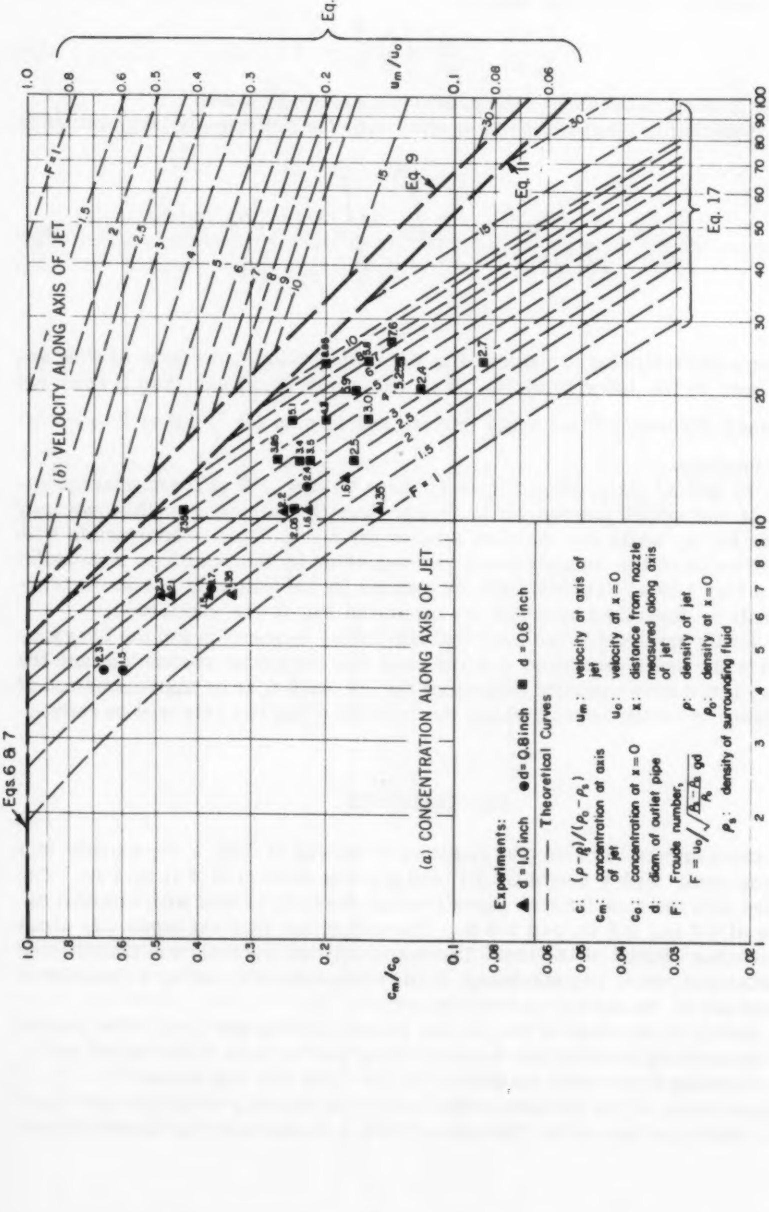

FIG. 1.—CONCENTRATION AND VELOCITY ALONG AXIS OF JET

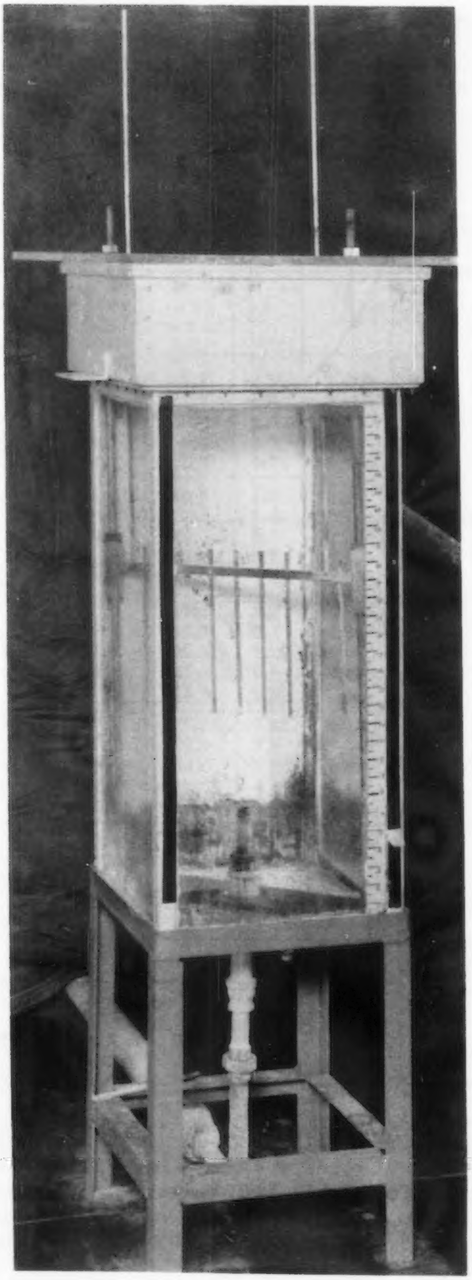

FIG. 2.—EXPERIMENTAL SETUP

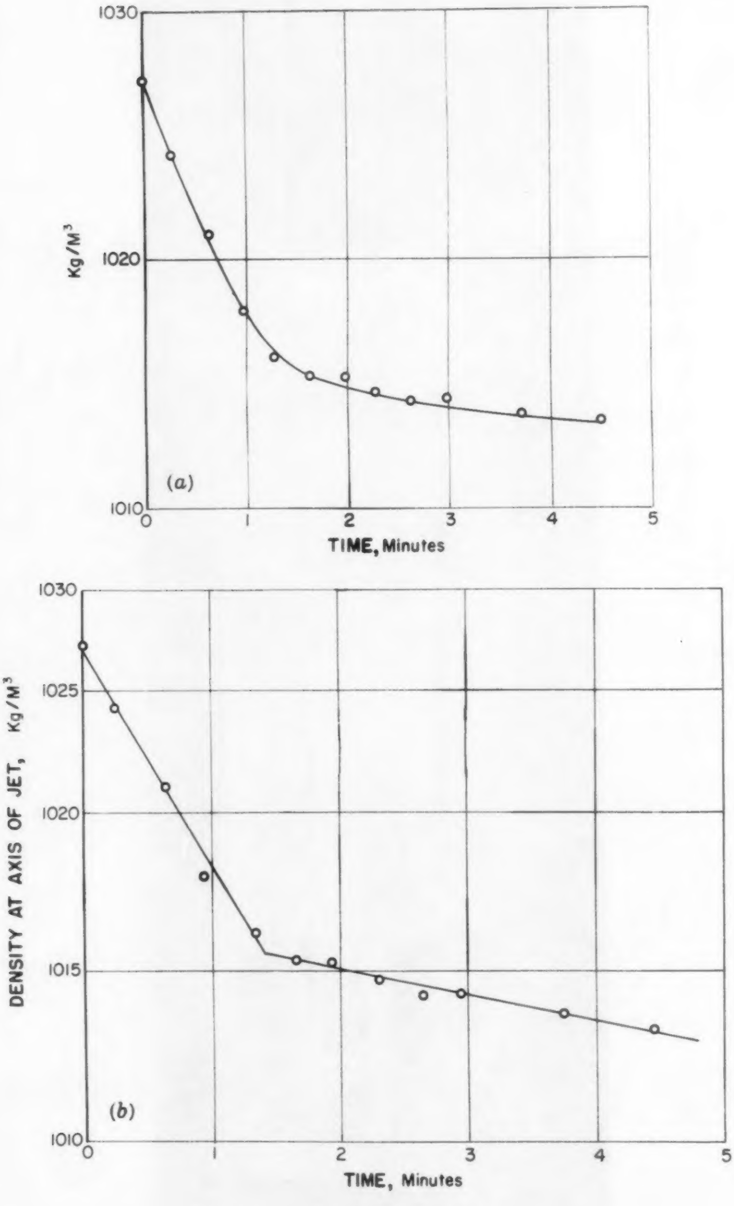

FIG. 3.—TIME HISTORY OF DENSITY

were taken at different points in the jet through pipes with an internal diameter of 3/16 in. The pipes were mounted to a frame that could be moved through the fluid in such a way that the inlet opening described a vertical line (Fig. 2). The discharge through the pipes was such that a period of 25 sec was required to collect a sample of 100 cc. The density of the samples was measured with a Westphal balance. The balance was accurate to about 0.1 kg per cu m.

TABLE 2.—RESULTS OF MODEL RUNS

Run	d, in inches	$\frac{x}{d}$	u_o, in cm per sec	$\rho_s - \rho_o$ in kg per m	F	R	$\frac{c_m}{c_o}$	Theory $\frac{c_m}{c_o}$ theory	Theory deviation, in % of c_o
21	0.6	10.5	16.4	42.5	2.05	2,060	0.24	0.235	+ 1/2
23	0.6	20.4	19.2	41.7	2.4	2,400	0.12	0.10	+ 2
15	0.6	13.8	16.8	30.0	2.5	2,100	0.17	0.175	- 1/2
16	0.6	23.8	21.0	39.8	2.7	2,620	0.085	0.08	+ 1/2
17	0.6	23.8	18.1	29.7	2.75	2,280	0.085	0.085	0
22	0.6	17.3	21.2	34.5	3.0	2,670	0.16	0.15	+ 1
8	0.6	13.8	22.0	27.8	3.4	2,760	0.23	0.215	+ 1-1/2
9	0.6	13.8	20.2	22.1	3.5	2,540	0.22	0.22	0
6	0.6	10.5	27.2	39.2	3.6	3,420	0.345	0.33	+ 1-1/2
10	0.6	13.8	30.8	43.2	3.85	3,860	0.26	0.23	+ 3
3	0.6	17.3	23.0	19.7	4.3	2,900	0.20	0.19	+ 1
5	0.6	17.3	37.0	34.7	5.1	4,640	0.24	0.21	+ 3
12	0.6	23.8	32.5	25.5	5.25	4,070	0.135	0.135	0
1	0.6	23.8	32.1	20.3	5.8	4,030	0.16	0.14	+ 2
2	0.6	20.4	30.6	18.5	5.9	3,840	0.17	0.175	- 1/2
7	0.6	10.5	47.1	27.2	7.35	5,900	0.43	0.49	- 6
46	0.6	26.6	47.5	26.0	7.6	5,950	0.14	0.14	0
43	0.6	23.2	69.0	32.0	8.85	8,650	0.20	0.19	+ 1
44	0.6	26.6	67.8	32.5	9.75	8,500	0.185	0.17	+ 1-1/2
36	0.8	4.4	10.8	26.2	1.5	1,770	0.60	0.57	+ 3
38	0.8	6.9	11.6	23.5	1.7	1,920	0.37	0.37	0
35	0.8	4.4	17.2	27.0	2.3	2,780	0.67	0.76	- 9
42	0.8	11.9	17.9	27.7	2.4	1,800	0.22	0.18	+ 4
47	1.0	6.6	12.8	35.8	1.35	2,660	0.33	0.32	+ 1
51	1.0	10.6	13.2	39.1	1.35	2,740	0.15	0.17	- 2
48	1.0	6.6	14.7	39.8	1.5	3,060	0.375	0.35	+ 2-1/2
52	1.0	10.6	16.1	40.7	1.6	3,360	0.22	0.195	+ 2-1/2
54	1.0	12.6	16.3	41.7	1.6	3,380	0.18	0.155	+ 2-1/2
49	1.0	6.6	20.4	36.3	2.1	4,250	0.46	0.45	+ 1
53	1.0	10.6	20.4	34.3	2.2	4,250	0.25	0.245	+ 1/2
50	1.0	6.6	24.5	46.5	2.3	5,100	0.49	0.47	+ 2

The time history of the density of the fluid of the jet was measured at different points in the jet. A typical example is shown in Fig. 3. Fig. 3(a) shows the time history on ordinary scale and Fig. 3(b) shows the same data on a semi-logarithmic scale. The first steep part of the logarithmic plot (and the corresponding part of the other plot) describes the establishment of equilibrium.

The second less steep part describes how the equilibrium is disturbed by the fact that the surrounding fluid is gradually mixed with the fluid of the jet. The small dimensions of the tank made it impossible to obtain a real equilibrium. Therefore, the density corresponding to the point of intersection of the two straight lines of the logarithmic plot has been considered to be the equilibrium density. Combined with any error made in the density measurement itself,

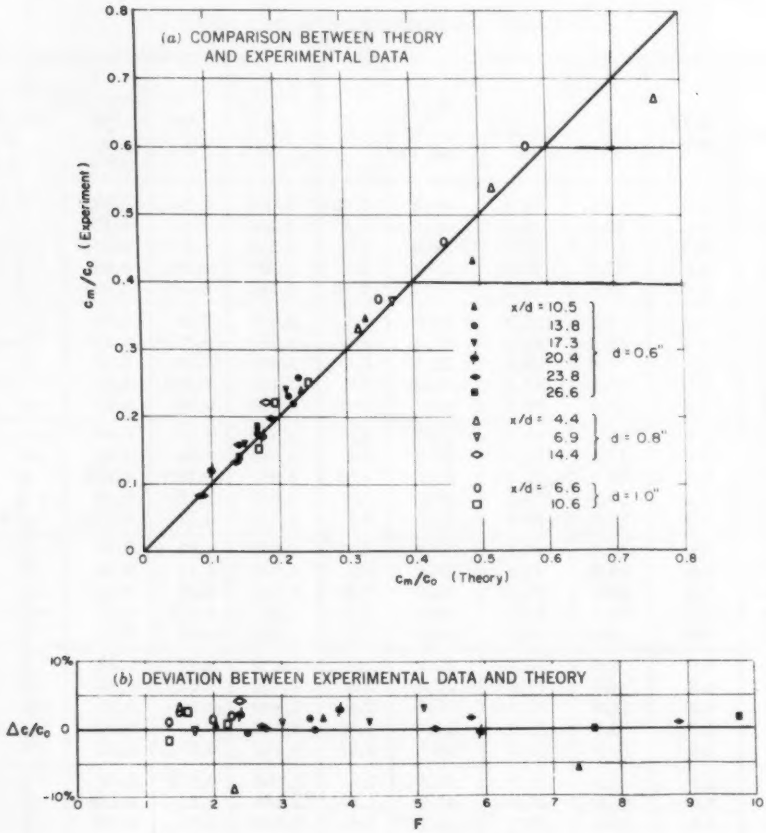

FIG. 4.—COMPARISON BETWEEN THE EXPERIMENTAL RESULTS AND THE THEORETICAL DATA

this method will cause an error of about 0.5 kg per cu m in the determination of the density difference between the fluid of the jet and the surrounding fluid.

The dilution is expressed in percentage. It is defined as $\frac{c}{c_o} = \frac{\rho_s - \rho}{\rho_s - \rho_o} \times 100$. For the experiments, $\rho_s - \rho_o$ varied between 20 kg per cu m and 50 kg per cu m. Hence the possible absolute error in the determination of the dilution is about $\frac{0.5}{20 \text{ to } 50} = 2\%$.

The determining circumstances and the results of the model runs regarding the density are shown in Table 2. For most of the runs the Reynolds number is sufficiently high to assure turbulent flow conditions.

The experimental results are plotted in Fig. 1. A comparison between the experimental results and the theoretical data, based on the theory of Morton, et al.,[9] is shown in Figs. 4(a) and (b).

The experimental results agree well with theory. The approach, suggested by the equations of Schmidt, to combine the results of the non-buoyancy case with those of the buoyancy case yields a good description for the intermediate case. That Eqs. 16 and 17, due to Morton, et al.,[9] take a small initial momentum in account is the reason for the good agreement obtained for small values of F.

Measurements of the Velocity.—A pitot tube has been used in an attempt to measure the velocity-distribution in the jet. The velocity readings were, however, insufficiently accurate because of the following reasons: (a) equilibrium conditions were not reached in the tank during the model runs; (b) the velocity in the jet is a function of the distance from the nozzle, while a pitot tube requires a constant velocity between the static- and dynamic-opening; and (c) the small inaccuracies made in the determination of the velocity-head were important because of the magnitude of the velocities involved (20-30 cm per sec).

Since no other instrument (small propellor, for example) was available, no further attempt to measure the velocities has been made.

CONCLUSIONS

The family of curves shown in Fig. 1(a) describes the dilution due to a submerged circular turbulent jet, issuing vertically upward in a fluid with a higher specific gravity than the jet. The curves are based on theory, which is adequately confirmed by model experiments. The density-range covered by the experiments is such that the curves may be used when sewage disposal in the marine environment is considered.

The family of curves, shown in Fig. 1(b), describes the velocity-distribution in the jet. The curves are based on theory. No attempt has been made to confirm this set of curves experimentally.

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Journal of the
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TRANSLATIONS OF FOREIGN LITERATURE ON HYDRAULICS

Second Progress Report of the Task Force on List of Translations of the Committee on Hydromechanics of the Hydraulics Division

INTRODUCTION

In an attempt to bring to hydraulic engineers an up-to-date listing of translations of foreign literature on hydraulics, it is the intent of the task force preparing this report to issue, from time to time, progress reports containing suitable items. Interested readers are urged to submit discussions to (a) add to this list, (b) offer suggestions for improvement, and (c) otherwise assist the task force in fulfilling its aims. The present list is the second issued since the publication, in 1957, of ASCE Manual 35 entitled "A List of Translations of Foreign Literature on Hydraulics." The first list appeared in the August 1959 Journal of the Hydraulics Division. This list is to be considered as an addendum to Manual 35 and will be included in a revision of that Manual when the time is deemed appropriate.

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Note.—Discussion open until November 1, 1960. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 6, June, 1960.

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Respectfully submitted,
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APPENDIX—

LIST OF ABBREVIATIONS

AEC	Atomic Energy Commission, Washington, D. C.
ASTIA	Armed Services Technical Information Agency, Dayton, Ohio
ATS	Associated Technical Services, Inc., East Orange, N. J.
BAM	British Air Ministry - See PKL
DSIR	Department of Scientific and Industrial Research, London, England
DTMB	David Taylor Model Basin, Navy Department, Washington, D. C.
ESL	Engineering Societies Library, United Engineering Trustees, Inc., New York, N. Y.
LC	Library of Congress, Washington, D. C.
LLU	Lending Library Unit, London, England
NACA	National Advisory Committee for Aeronautics, Division of Information, Washington, D. C. (Name changed to NASA)
NASA	National Aeronautics and Space Administration, Technical Publications Announcements, Washington, D. C.
NBS	National Bureau of Standards, Fluid Mechanics Section, Washington, D. C.
ONI	United States Office of Naval Intelligence, Technical Translations, Department of Commerce, Office of Technical Services
OTS	Office of Technical Services United States Department of Commerce, Washington, D. C.
PKL	Paul Kollsman Library, Institute of Aeronautical Sciences, New York, N. Y. (Holds BAM Translations listed)
SAF	St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis, Minn.
SLA	Special Libraries Association, Translation Pool, John Crerar Library, Chicago, Ill.
TM	Technical Memorandum
Tr.	Translation
Tr.by	Translated by
TVA	Tennessee Valley Authority, Knoxville, Tenn.
U. of C.	University of California, Department of Engineering, Berkeley, Calif.
USBR	United States Bureau of Reclamation, Office of Assistant Commissioner and Chief Engineer, Denver, Colo.
WES	U. S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.
VDI	Verein Deutscher Ingenieure

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FLOW LOSSES IN THE LOWER GILA RIVER

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SYNOPSIS

This paper describes a method for estimating losses from infrequent flows in the lower reaches of the Gila River, in Arizona. The same procedure could probably be applied to other intermittent streams if sufficient data are available.

INTRODUCTION

The usual procedure for studying or analyzing flood flows in a stream wherein water flows continuously is some kind of mass-curve technique. The literature concerned with this problem is voluminous. However, such procedure has little application to intermittent streams because the available data are too discontinuous to provide a suitable universe for common statistical methods.

Because of the discontinuous nature of the gaging records, the only rational way to study flood flows of an intermittent stream is to take them one at a time, not enmasse. At a gaging station on an intermittent stream, there will be periods of weeks, months, even years, when no flow will be recorded. Because of the many zeros in the record any sort of average is meaningless. Any given zero flow may reflect a condition such that the last surface water may have disappeared only a few feet, or many miles, above the gage. If there is another gaging station upstream, it is obvious that the upper gage will record

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many more daily flows than the lower gage, provided that local inflow between the gages is negligible or absent.

It is possible to express the percentage of loss between an upstream and a downstream station in terms of the quantity of flow at the upstream station and duration of flow at the downstream station. The procedure is to cumulate concurrent flow quantities at both stations, making proper allowance for time of travel between the two. This allowance can most conveniently be made by matching the days of peak flows, but is not constant. When the streambed has been dry for a long time, much water will be required to fill surface depressions and to initiate infiltration. These requirements take time as well as water. When flow has been established, the time of travel may be reduced to as little as a third of the time of travel for a dry streambed.

As flow continues, the percentage of water lost between the upper and lower stations changes, rapidly for the first few days and then more slowly, approaching a limit. If flow continues long enough to indicate the value of the limit, it may be inferred that this value is primarily caused by evaporation, since equilibrium will have been approached between surface flow and infiltration. Higher percentages of loss represent the prior mentioned requirements for filling depressions and initiating infiltration.

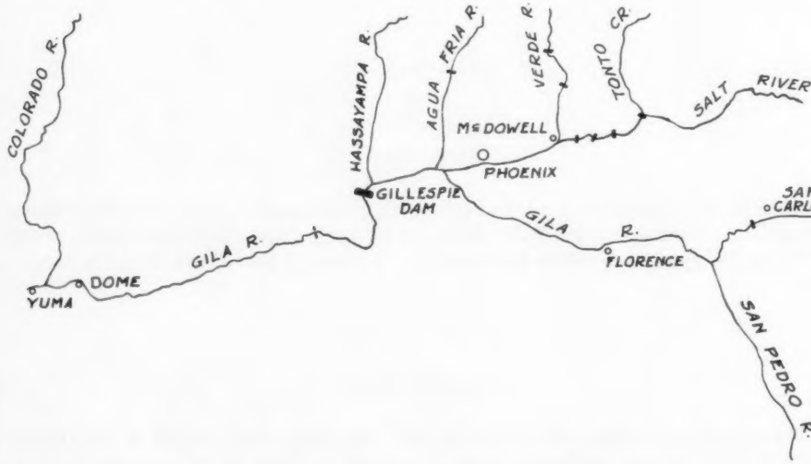

FIG. 1.—LOCATION MAP

GILA RIVER BASIN

An important intermittent stream of southwestern United States is the lower portion of the Gila River, in Arizona (see Fig. 1). The reaches considered herein are from Dome to Gillespie Dam, since 1928, and from Dome to McDowell and San Carlos, for 1904 and 1905.

The Gila River Basin covers an area of 58,180 sq miles, of which about 88% is in Arizona, 10% is in New Mexico, and 2% is in Sonora. The upper

reaches of the Gila River and its principal tributaries are in mountainous terrain within which are several peaks of over 10,000 ft elevation. It is from this terrain that nearly all runoff is derived. At about 250 to 300 miles above the mouth of the Gila River, several storage dams have been constructed to impound this runoff. The larger dams are: Coolidge on the Gila; Roosevelt, Horse Mesa, Mormon Flat, and Stewart Mountain on the Salt; Horseshoe and Bartlett on the Verde; and Carl Pleasant on the Ague Fria. In 1952, the usable storage capacity of reservoirs on the Gila River and its tributaries was 3,446, 000 acre-ft[2]. This storage capacity is not quite twice the average annual inflow to the reservoirs.

Downstream from the reservoir system is the Salt River Valley containing some 800,000 acres of irrigated land and several cities and towns, the largest of which is Phoenix. In nearly every year, all available surface water released from storage or running in unregulated tributaries is wholly con-

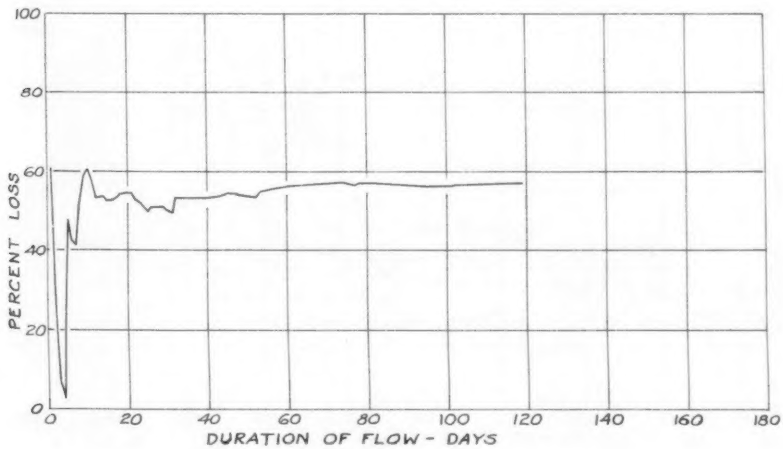

FIG. 2.—FLOW AT GAGE FROM JULY 30, TO NOVEMBER 26, 1904

sumed by irrigation and domestic uses. In addition, a large quantity of ground water is pumped and consumed. The result is that very little water appears below Gillespie Dam, the outlet of the valley, and only rarely does any water reach the mouth of the Gila River or the gaging station at Dome, 12 miles above the mouth. The river distance from Gillespie Dam to Dome is about 150 miles. Many of the flows registered at the Dome gage originate from infrequent rains falling west of Gillespie Dam or an return flows from irrigated areas between the two gages.

Early Records[3].—For a brief period during the years 1904, and 1905, gaging stations were operated concurrently on both the Salt and Verde Rivers just above their junction at McDowell, and on the Gila River at San Carlos (near

[2] "Ground Water in the Gila River Basin and Adjacent Areas—A Summary," An open file report of the USGS, 1952.

[3] USGS Water Supply and Irrigation Paper No. 133.

present Coolidge Dam) and at Dome, also called Gila City. During this period, there were no storage reservoirs in the basin.

The Gila River was dry at Gila City (Dome) from January 1, 1904, (and for an unknown time earlier) to July 29, 1904. Flow at the gage started July 30, and continued until November 26, after which date the river was again dry. Only gage heights are published for Dome, but a rating curve was prepared by the author, using data for other periods, to estimate the average daily flows. Average daily flows are published for the three upstream stations.

The apparent time of travel of flood peaks, after flow was established at Dome, was seven days from McDowell to Dome, and five or six days from San Carlos to Dome. While the flow passing Dome persisted for 119 days, at no time did it become large. The maximum daily flow at Dome was 4,560 day-sec-ft and the total flow was about 220,000 acre-ft. The combined flows passing the three upstream stations was about 508,000 acre-feet. The difference,

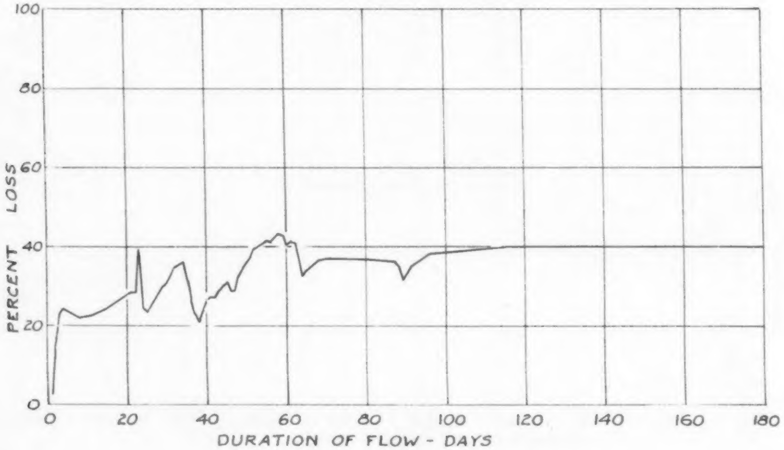

FIG. 3.—FLOW FROM JANUARY 16 TO JULY 19, 1905

or apparent loss, was 288,000 acre-ft. This difference ignors both unmeasured inflows and consumptive uses, neither of which can be estimated with any assurance for this period.

Flows at McDowell and at San Carlos were combined at six days earlier than Dome. The combined flows were cumulated day by day and the same cumulation was made for the flows at Dome. The cumulative differences were considered to represent the cumulative losses and were tabulated both in day-sec-ft and as percentages of the combined upstream flows. Percentage of loss to the end of the flow period was 56.8%. Cumulative loss percentages between 55% and 57% were consistent for more than the last half of the period (see Fig. 2). It is concluded that this order of loss percentage fairly represents the conditions existing in 1904, with a relatively low but persistent flow.

The Gila River was dry at the Dome gage from November 26, 1904, until January 16, 1905. Flow began on that date and continued until July 19, 1905, except for February 2, 3, and 4. Very large flood peaks occurred in this six

months period, reaching maxima of 82,000 day-sec-ft on February 8, 95,000 day-sec-ft on March 20, and 64,000 day-sec-ft on April 14. Except for a relatively small peak of 13,000 day-sec-ft on April 28, flows gradually decreased from April 14, to July 19, when the river was again dry at Dome. Because of the numerous large flood peaks, the calculated cumulative percentage of loss is not as constant as was that calculated for the 1904 period (see Fig. 3). For the last half of the period the loss was fairly steady at around 40%, but during the first half the loss varied from 21% to 43%. The measured inflow for the period was about 5 million acre-ft and the outflow at Dome was about 3 million acre-ft. Again, consumptive uses and unmeasured inflows were ignored.

Later Records[4].—From 1907, until a recording gage was established in May, 1929, there are no records of discharge at Dome. A recording gage had previously been established at Gillespie Dam in July, 1924. From May, 1929,

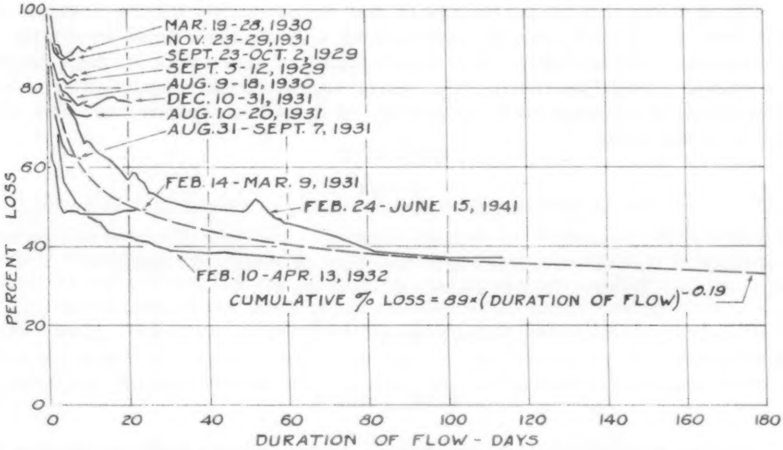

FIG. 4.—GILA RIVER, 1929 TO 1955

through September, 1955, there were a total of 312 months. In only 29 of those months were there recorded flows at Dome in excess of 1,000 acre-ft a month. For the same period there were 90 months when the flow past Gillespie Dam exceeded 1,000 acre-ft. Therefore, two-thirds of the months when there was appreciable flow past Gillespie Dam there was no flow at Dome. Under these circumstances, a correlation of monthly flows at the two stations is quite meaningless and a correlation of annual flows is impossible.

During the 27 yr (1929-55) of records studied on the Gila River, there were only eleven occasions when flows passing Gillespie Dam reached the Dome gage for sustained periods of seven days or more. The longest periods of sustained flow were in 1941, for 114 days, and in 1932, for 66 days. Two flows, both in 1931, continued for 22 days and 25 days. Seven other flows were sustained for 7 days to 12 days.

[4] USGS annual water supply papers for the Colorado River Basin.

It was found that the percentage of the accumulated flow passing Gillespie Dam, but lost before reaching Dome, was related to the duration of flow at Dome. It was also found that the time of travel was affected by the antecedent condition of the streambed. If the bed had been dry for several months, the travel time was as much as eleven days. After the channel had been wetted, the time was reduced to three or four days. The time of travel for each period or sub-period was determined by matching the appearances of flood peaks at the two gaging stations. The differences between the cumulative flows at Gillespie Dam and the cumulative flows at Dome were found and expressed as percentages of the Gillespie flow. These differences were considered to be the cumulative losses in the reach, ignoring any inflow caused by local precipitation. The determination of any such local inflow would involve the roughest kind of estimates, as there are no records of flow and an insufficient number and inadequate distribution of rainfall records to make useful rainfall-runoff correlations possible. Consumptive uses were small and were ignored. The results were plotted as Fig. 4.

Fig. 4 shows that the percentage of loss drops quickly for the first few days and then flattens out, usually approaching a limit, sometimes gradually and sometimes quite abruptly. For the two flows of more than 30 days duration the curves are of hyperbolic form, which suggested the calculation of an empirical equation to represent the average of these two occurrences. The equation is of the form:

$$y = ax^{b} \qquad (1)$$

in which y is cumulative percentage loss, a is the first day percentage loss, x is days of flow at the lower gage, and b is an empirical exponent. For the reach from Gillespie Dam to Dome the equation is:

$$y = 89\, x^{-0.19} \qquad (2)$$

CONCLUSION

It seems probable that an equation of the same form could be derived for any other intermittent stream if sufficient data were available. The constants would undoubtedly be different.

For those streams wherein flows are more frequent and of longer average duration than for the lower section of the Gila River, a relationship linking duration of flows to channel losses, or to flows at the lower station, would have more significance. If the economic value of the water could be established it would then be possible to determine to what extent salvage should be undertaken, by storage or channel control, to increase the duration of flows and thereby decrease the cumulative percentage of loss.

ACKNOWLEDGEMENT

The Gila River studies were made by the author while he was employed by the Colorado River Board of California. Fig. 4, with an explanation of its derivation, was presented to the United States Supreme Court in the case of Arizona vs. California, et al, by C. C. Elder, F. ASCE, Hydrographic Engineer, the Metropolitan Water District of Southern California.

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

CAPACITANCE METHOD OF MEASURING WATER FILM THICKNESS[a]

By R. H. Black[1]

SYNOPSIS

A water film thickness gage was developed for making rapid measurements of the depth of a water sheet flowing on a plane surface. The gauge uses the electrical capacitance between a fixed metal plate and the water surface to sense the air gap and, by use of differences, measure the water depth.

Rapid readings of the depth of a disturbed water sheet flowing down a plane were made. The dynamic range of the instrument was 0.130 in. with an accuracy of ± 0.003 in. Transverse roller waves 3 in. long were resolved with this instrument; through the use of a special "probe," waves 1/2 in. long were resolved.

INTRODUCTION

The U. S. Naval Radiological Defense Laboratory is currently studying the transport of radioactive particulate contaminants by water sheets. Stream depth is one of the most important factors in the equations describing sheet-flow of water on an incline plane.[2] This depth can be measured precisely and accu-

Note.—Discussion open until November 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 6, June, 1960.

[a] The opinions expressed by the author do not necessarily reflect those of the Navy Dept.

[1] Chem. Engr., Aerojet-General Nucleonics San Ramon, Calif. Formerly with the U. S. Radiological Defense Lab., San Francisco, California.

[2] "Transport of Contaminant by a Water Film," by R. H. Heiskell, R. H. Black, and H. L. Burge, U. S. Naval Radiological Defense Lab. Review and Lecture, USNRDL-RL-86, February, 1958.

rately to ±0.001 in. by the use of a micrometer point gauge when stream flow is uniform and the surface is glassy smooth. However, when water flows down an incline plane with a slope greater than 0.01, pronounced transverse roller waves are generated a short distance from the entrance to the plane, and these waves introduce serious problems in the use of a point gauge for measuring depths. Wave height is not a constant for a given flow condition and trough measurements are questionable, as the "end point" for determing contact is subjective. Furthermore, no information concerning wave-shape results from the use of this techique.

A review of the literature revealed two other methods of determining water sheet thickness. These were an optical method,[3] which requires the water to be flowing on the outside of a cylinder, and a capacitance method,[4] utilizing a frequency-modulated carrier. The first method was impossible due to the unsuitable geometry of the apparatus required to be used and the second was considered likely to be subject to electronic drift.

As future progress in the experimental study of sheet flow depends partly on the development of an instrument that can be used for making depth measurements during non-uniform flow, the objective of this project was to develop an instrument capable of making depth measurement of a sheet of water flowing over a plane surface, and to obtain adequate response to demonstrate wave shape.

APPROACH

There were several potentially useful means for determining the thickness of a water sheet, grouped under two basic approaches. The first was to utilize absorptive properties relating to the thickness of the water sheet by (a) detecting the absorption of beta radiation passing through the water, or (b) detecting the absorption of light passing through dyed water. The second basic approach was to measure the distance to the air-water interface and obtain the thickness by (a) subtracting the distance to the stream bed by detecting contact with the surface electrically or optically, (b) detecting the beta backscatter of a radioactive source fixed above the surface, or (c) detecting the electrical capacitance between a metal plate fixed above water and the (electrically grounded) water surface.

A brief consideration of these possible means from the viewpoint of utilizing readily available equipment revealed that most of them had shortcomings. Methods dependent on radioactive sources would require relatively expensive equipment and heavily shielded radiation sources. Electrical and optical-contact detection equipment was not available. Light-absorption methods, utilizing dyed water, would necessitate a recirculating system and a translucent or transparent stream bed, and would complicate or delay other studies requiring a transparent water sheet. However, a capacitance transducer was commercially available utilizing a patented mechanic-electric transducer which had a specified sensitivity of 2 v output per μfd change of capacitance and less than 0.1% drift per.[5] The detection of electrical capacitance between a fixed ref-

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[3] "Heat Transfer by Condensing Vapors," by C. G. Kirkbride, Transactions, Amer. Inst. of Chem. Engrs., 30:170, 1934.

[4] "Characteristics of Flow in Falling Liquid Films," by A. E. Dukler and O. P. Bergelin, Chem. Engr. Progress, Vol. 48, No. 11, 1952, p. 557.

[5] "Mechanical - Electric Transducer," by Kurt S. Lion, Rev. Sci. Instruments, 27:222, 1956.

erence plate and the water surface would avoid the disadvantages of contact with the water or dyeing of the water.

APPARATUS

The commercially available capacitance transducer is essentially a capacitance comparator and gives a direct current (D. C.) signal that is proportional to the percentage difference between the values of two capacitors with a common ground.[6] The transducer is excited by and permanently connected to a 250 kc carrier generator. One of the capacitors, C^1, is a plate, or the "probe," fixed above and parallel to the electrically grounded water. The other, C^2, is an adjustable reference capacitor. The relative positions of these and the other electrical units are shown in Fig. 1. They are described in the Appendix. The R. F. filter and D. C. power supply were constructed according to specifications

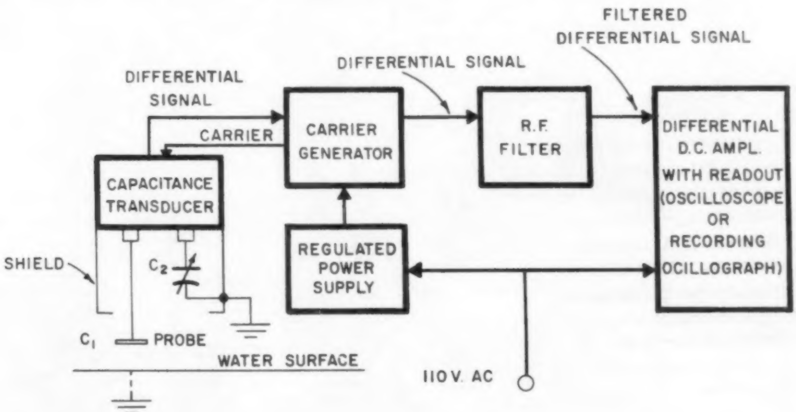

FIG. 1.—BLOCK DIAGRAM OF INSTRUMENT

of the manufacturer of the capacitance transducer. The D. C. amplifier and readout can be either an oscilloscope with a camera or a recording oscillograph.

The probe area was arbitrarily chosen as 1.0 sq in. (6.42 sq cm). Capacitor C_1 is designed to vary $\pm 0.3\,\mu$fd with water-sheet thickness changing by 0.130 in. (0.330 cm), and to have a mean capacitance, including fixed stray capacitance, of 3 to 5μfd. Capacitor C_2 is adjustable from 3 to 12μfd. The capacitance between the probe and the water surface is considered to be the same as that between two parallel plates as a first approximation in setting the probe parameters;

$$C_1 = 0.0885 \frac{s}{d} \qquad \text{. . . . . (1)}$$

in which C_1 is the capacitance (μfd), s denotes probe area (cm^2), and d is the distance from probe to water (cm)

With the values given previously, two equations can be established:

$$C_1 = 0.0885 \times \frac{6.42}{d} \quad \dots\dots\dots\dots\dots\dots\dots (2)$$

$$C_1 + 0.3 = 0.0885 \; \frac{\times\ 6.42}{(d + 0.330)} \quad \dots\dots\dots\dots\dots\dots (3)$$

Solving these equations gives 0.6μfd for C_1 and 0.38 in. (0.97cm) for d.

The probe (which is removeable) and a shield, that encloses and fixes the stray capacitance of the transducer, reference capacitor, and probe lead wires

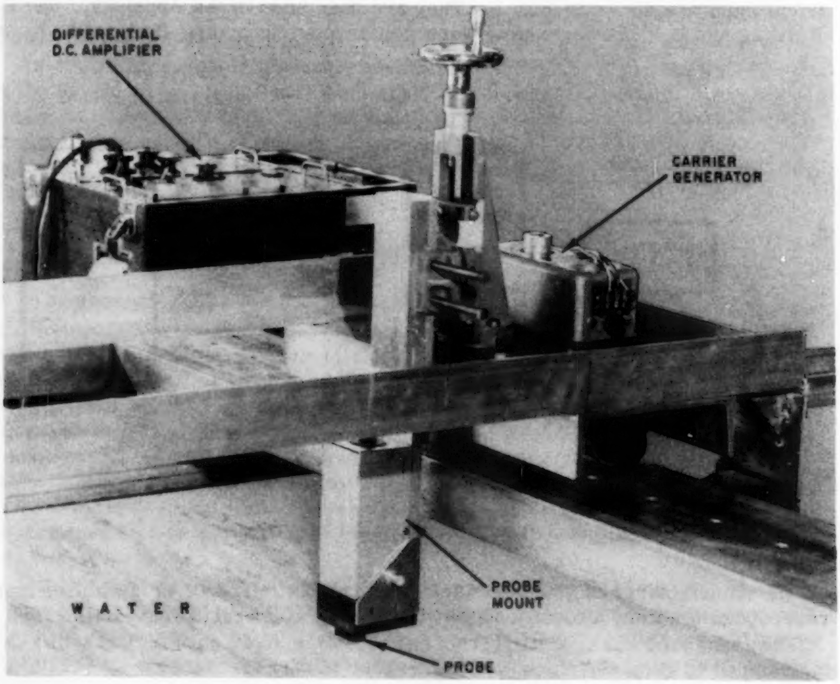

FIG. 2.—APPARATUS

are rigidly fastened to the probe mount. This assembly has been offered recently as a unit by the transducer manufacturer. The probe mount is held in a vise which rides on a vertical calibrated micrometer screw. With this micrometer screw, the probe can be raised or lowered by a known distance from the stream bed. The instrument is shown in Fig. 2.

CALIBRATION AND PERFORMANCE

A reasonable compromise between adequate sensitivity and an acceptable dynamic range was found with a probe 1-1/4 in. wide by 3/4 in. long, designated as the "standard" probe. With the standard probe set 0.400 in. above the stream bed, and the differential D. C. amplifier gain set to give full-scale response to a 1-v signal, the dynamic depth range is 0.030 to 0.160 in.

The maximum sensitivity of the instrument for measuring change of depth is approximately 0.0003 in. This sensitivity can be used for measuring water surface undulations; however, accuracy is not great enough to use the full sensitivity effectively in determining water film thickness. The sensitivity of the instrument utilizing the 0.130 in. dynamic range is +0.001 in.

The instrument was calibrated in place with the water flowing. A 4 in.-by-4-in. metal plate of known thickness (and approximately the thickness of the

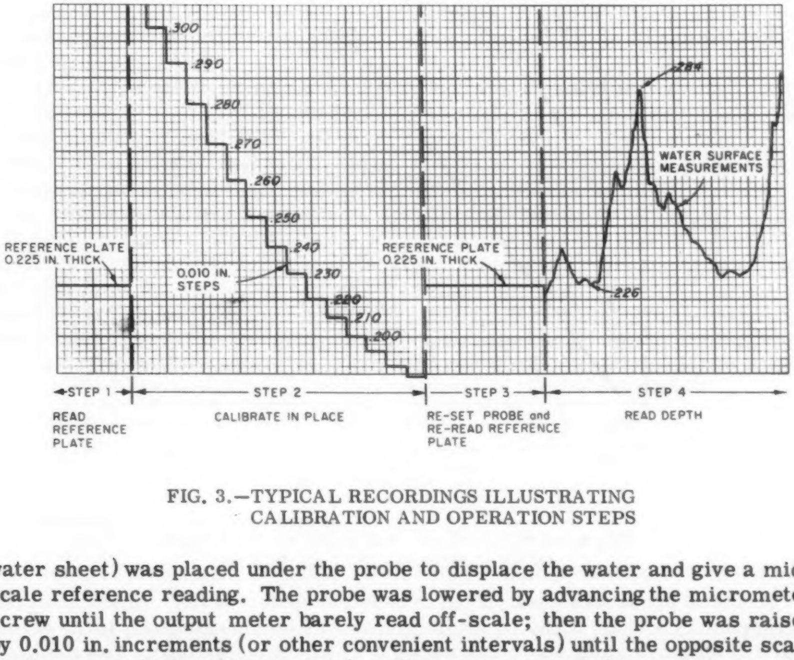

FIG. 3.—TYPICAL RECORDINGS ILLUSTRATING CALIBRATION AND OPERATION STEPS

water sheet) was placed under the probe to displace the water and give a mid-scale reference reading. The probe was lowered by advancing the micrometer screw until the output meter barely read off-scale; then the probe was raised by 0.010 in. increments (or other convenient intervals) until the opposite scale limit was reached, and incremental readings were recorded to provide a scale. Following this, the probe was re-set to the mid-scale reading, and the reading for the plate was recorded to provide a reference, thus, completing the calibration. The reference plate was removed, and the instrument was then measuring the water sheet thickness. Fig. 3 shows a specimen of the calibration data and depth data.

The accuracy of the instrument was determined by comparing its readings with water depth measurements of smoothly flowing water taken with a micrometer point gauge. Agreement was + 0.003 in.

The resolution of the instrument to longitudinal waves using the standard probe was measured by replacing the water sheet with a flat metal plate. This plate had several widths of slots cut to a depth of 0.030 in. to represent "waves." This plate was moved under the probe, in place of the water sheet, and the slot depth, as indicated by the instrument, was recorded. The results of this test are given in Fig. 4. A 0.5 in. slot can be resolved with a 50% response to amplitude, and a 1.5 in. slot is the shortest that can be resolved with no loss in amplitude. This resolution is assumed to be the same as the instrument resolution to longitudinal water waves.

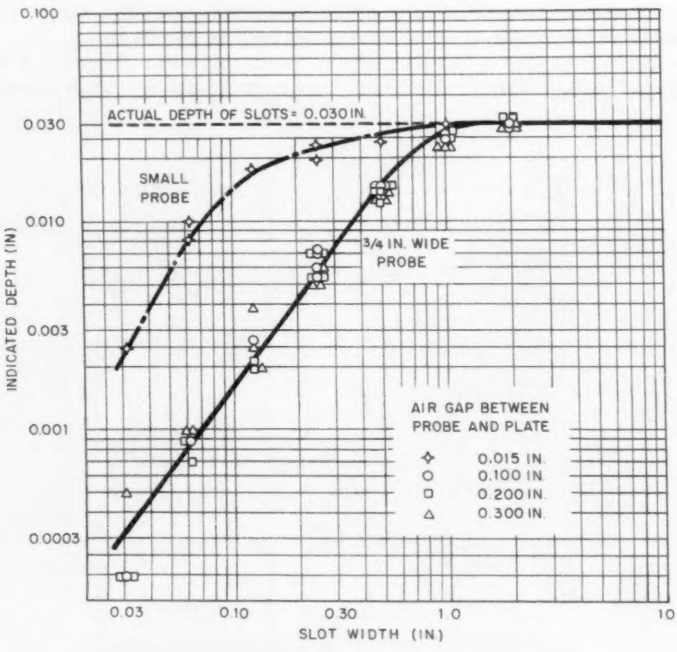

FIG. 4.—RESPONSE OF INSTRUMENT TO 0.030-in. DEEP SLOTS IN A METAL PLATE

A special probe was constructed to resolve shorter waves. This small probe is essentially a short 0.080 in. diameter rod fixed perpendicularly to the stream bed and shielded along its length with a 0.25 in. diameter tube. The results of resolution tests for this smaller probe are also presented in Fig. 4. A 0.1 in. slot can be resolved with a 50% response to amplitude. This probe has low sensitivity and very nonlinear response to water-sheet thickness, and, therefore, was not generally used for thickness measurements. However, it can be calibrated in the same manner as the standard probe.

CONCLUSIONS

The accuracy of the instrument, as determined by comparison with the micrometer point gauge, is limited by the conductivity of the water. Although the

impedance of the transducer is high relative to that of the water, slight fluctuations in the conductivity of the water due to depth and ion content affect the resistance and, therefore, the apparent "location" of the grounded side of C_1. Simple tests indicated that accuracy can be improved to ± 0.001 in. by increasing salinity to approximately 0.01 Normal NaCl.

The resolution of the instrument to waves is, in part, limited by the dimensions of the probe. The shortest logitudinal wave that can be resolved is equal to the length of the probe.

When the small probe is used, the resolution of waves is limited by divergence of the electrostatic lines of force from the small area of the probe to the water surface. If the electrostatic lines could be made parallel by applying the proper voltage to an electrostatic ring around the probe, then, the resolution of the instrument would be limited only by the smallest probe which could supply a readable signal.

The instrument sensitivity to changes in water depth is limited by the electrical noise level (which is approximately 2 millivolts), the maximum probe area as determined by wave resolution requirements, and the minimum air gap between the probe and the water surface, as determined by the anticipated fluctuation in depth.

Instrument stability is excellent; drift was within ± 0.001 in. over a 4-hour period after a 15 min. initial warm-up. The transducer life appears reasonable; the unit required repair after one year of modest use.

Instrument sensitivity, useful range, and wave definition are interrelated, as indicated above, must be balanced against each other for a particular application. By manipulating the value of C_2, the area and shape of the probe, the distance of the probe from the mean water surface, and the type of readout or recording equipment, this depth-measuring instrument can be applied to a variety of hydraulics studies involving a free liquid surface. The rapid indication of water surface and ease of reading, even with a disturbed surface, can save much time. The availability of electrical signal output suggests the possibilities of centralized simultaneous recording of multiple stations and automatic control.

APPENDIX

Equipment used for this report:

Capacitance Transducer; Delta Unit Model 901-1, Decker aviation Corp., Philadelphia, Pa., 2V/μfd sensitivity at 10 μfd initial capacity.

Regulated Power Supply; NRDL design (vacuum tube regulation) to comply with specifications furnished by manufacturer of capacitance transducer.

Filter; NRDL-fabricated per diagram supplied in capacitance transducer instruction manual.

Direct-writing Oscillograph; Sanborn, Model 128, 200 mv/cm sensitivity, differential D. C. input.

Oscilloscope; Dual Beam, Type 322A, A. B. DuMont Lab., Clifton, N. J. 200 mv/cm deflection sensitivity, differential D. C. input (one beam required).

Oscillograph-record Camera; Type 297, A. B. DuMont Lab., Clifton, N. J.

Constant Voltage Transformer; Solavolt No. 50106, 500 Watt.

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Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

SEDIMENTATION ASPECTS IN DIVERSION AT OLD RIVER

By Fred B. Toffaleti[1]

SYNOPSIS

In recent years the Old and Atchafalaya Rivers have diverted a progressively increasing percentage of Mississippi River flow. The damaging consequences posed by this continuing action necessitated controlled diversion. This paper presents the sedimentation aspects of channel evolution leading to necessity of control. It is shown that the distributary system is in a state of imbalance in sediment transport capacity, the nature of which led to the conclusion that the flow capacity of the Atchafalaya River can be expected to increase annually regardless of the type of water year. This conclusion is substantiated by field observations at the Simmesport discharge range.

INTRODUCTION

The Atchafalaya River, which is connected with the Mississippi River by Old River, is the farthest upstream distributary of the Mississippi River. Fig. 1 is a plan map showing the Mississippi River below latitude Natchez, the lower end of the Black and Red Rivers, the Atchafalaya River, and connecting Old River. From the head of Old River it is approximately 140 miles to the Gulf of Mexico by way of the Atchafalaya River. By way of the Mississippi River past Baton Rouge and New Orleans it is 320 miles, or more than twice the distance via the Atchafalaya River. Old River is approximately 6.5 miles in length.

Note.—Discussion open until November 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 6, June, 1960.

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The lower 60 miles of the Red River channel and the lower portion of Black River, whose principal flow is from the Ouachita River, are in a backwater basin containing numerous shallow lakes and low-level swamp areas. At the peak of Mississippi River floods, and coincident peak flows in the Atchafalaya River, the water-surface slopes through this backwater area are very flat. Consequently, the Red and Ouachita Rivers system can contribute only very fine sediments to the Atchafalaya River at high stages.

In recent years the Old and Atchafalaya Rivers have diverted progressively increasing percentages of Mississippi River flow. Studies of this development led to the conclusion that this diversion from the Mississippi River will reach a "critical" stage (approximately 40% of the major river's flow) by 1975. The consequences of such an eventuality necessitated a controlled diversion. Con-

FIG. 1.—RIVER SYSTEM BELOW LATITUDE NATCHEZ

struction of control structures was initiated in 1955 and is now essentially complete. All work, including new channels, bank protection, a lock, and the closure of Old River, is scheduled for completion by the end of 1963. Details of this project which include river history, the general problem, hydraulic requirements of control works, type structures and foundation design, have been presented previously.[2]

Previous discussions of this problem area have dealt mostly with flow and channel dimensional changes. This paper presents the sedimentation aspects of channel evolution leading to the necessity of controlled diversion. The sediment transport characteristics of channel reaches in the study area provide an

[2] "Old River Diversion Control: a Symposium," Transactions, ASCE, Vol. 123, 1958, pp. 1129-1181.

excellent example of sediment transport capacity imbalance in an interconnected system of rivers. For the purposes of this paper, the discussion will be confined to Old River and the upper 41 miles of the Atchafalaya River.

ATCHAFALAYA RIVER

The Atchafalaya River flows, for the most part, through the leveed Atchafalaya Basin Floodway which averages about 17 miles in width. At project design flood stage this floodway will be required to pass 1,500,000 cfs which is 50% of the project design flood flow at this latitude. The river channel is in the approximate center of the floodway and is leveed from Mile 5 to 53, measuring from its head. This leveed reach varies from 2,500 ft to 3,500 ft in width and is flanked on the west side by the West Atchafalaya Floodway and on the east side by the Morganza Floodway. From Mile 53 to 115, flow is channeled through low-lying swamp and shallow lake areas.

By virtue of a lesser distance to sea level, complemented by widespread flow in the lower basin, the water surface slope in the upper leveed reach at appreciable flow is steeper than that of the Mississippi River below Old River. Also, the leveed reach is rather deep in comparison with its width. For instance, in 1957 the bankfull stage width-to-depth ratio of the Atchafalaya River at Simmesport was 32 as compared to 120 for the Mississippi River just below the head of Old River. In Old River at Torras, 1.1 miles from the Mississippi River, it was 39. Mean depths measured at bankfull stage in 1957 were approximately 60, 50, and 45 ft, respectively, at Simmesport, Torras, and Red River Landing.

It cannot be stated as being of exact or uniform occurrence, but there is evidence that degradation occurs in the Atchafalaya River at any time of medium and higher stages coincident with low velocities in Old River. At these stages Red River sediment supply in the sand range is negligible because of flat slopes in the backwater area. Under any circumstance the sand fraction of Red River sediment is much finer than that coming from the Mississippi through Old River. Therefore, the important sediment supply which affects the regimen of the Atchafalaya River comes from Old River. Conversely, when there is appreciable flow in the Atchafalaya coming almost entirely from the Mississippi River, some aggradation of the Atchafalaya is to be expected.

Extreme cases of the latter condition are infrequent, the last occurrence being in June, 1943. At that time flow in the Atchafalaya was in excess of 400,000 cfs and all of it was coming from the Mississippi via Old River. Average velocities in Old River at that time reached as high as 8 fps as compared with less than 6 fps in the Atchafalaya. Roughly, this would make the supply of sediment to the Atchafalaya about 50% in excess of its transport capacity. Observations indicate that at bankful stage the mean bottom elevation at the Simmesport discharge range may raise or lower from 2 ft to 4 ft, depending on the percentage of Atchafalaya River discharge being contributed through Old River.

In 1952 a study was made of bed material transport capacity in the Atchafalaya River from its head to Krotz Springs (Mile 41). This study was based on 1949-50 channel conditions and water stages observed during the 1950 flood. These data showed that water surface slopes in this reach were not uniform either with respect to stage or location, nor was the channel section uniform in average width and depth. In the downstream direction the channel decreased

in width, increased in depth and, in general, there was an increase in water surface slope. It was indicated that the slope also increased with increasing stage.

To study the effects which these varying channel characteristics might have on sand transport, the river was divided into three reaches as shown in Fig. 2. This division of reaches, Barbre Landing to Simmesport, Simmesport to Melville, and Melville to Krotz Springs, was dictated by location of gaging stations at which daily observations are made.

Sand transport rating curves were then developed for each of these reaches in general accordance with the bed-load function theory developed by H. A. Einstein.[3] At that time the bed-load function theory was relatively new and initial studies were directed towards determining its applicability to the Atchafalaya River. Sediment data from frequent observations at the Simmesport discharge range were available for this purpose. It was found that application of the principles of the bed-load function theory, with a few modifications, would produce results in reasonable agreement with the measured sediment loads.

Fig. 3 shows the sand transport rating curves developed by this method for the three reaches. It is interesting to note that the rating curves show the leveed reach of channel to be, in general, in a state of equilibrium at flow up to 400,000 cfs. This is approximately bankfull flow in the leveed reach. Above 400,000 cfs, the transport capacity increases in the downstream direction. The difference between Reaches 1 and 2 is more or less constant, but in Reach 3 it is increasingly greater as the discharge increases. Thus, it is indicated that at that time there was sediment transport capacity imbalance in the upper reaches of the Atchafalaya River at stages above bankfull.

OLD RIVER

Old River is approximately 6.5 miles in length and is the connecting channel between the Atchafalaya and Mississippi Rivers. Its mean depth at bankfull stage is less than that of the Atchafalaya and somewhat greater than that of the Mississippi. Formerly its width was several hundred feet less than the Atchafalaya, but in recent years its rate of enlargement has exceeded that of the Atchafalaya and now there is little difference between the two, in this respect. The amount of sand supplied to the Atchafalaya River from outside sources includes that which passes into Old River from the Mississippi, plus and minus fill or scour in Old River, plus that contributed by the Red and Ouachita Rivers system. The latter system is not considered an important contributor of material that will affect the regimen of the Atchafalaya River.

Discharge through Old River is not entirely a function of stage; it is, rather, dependent on relative stages at its head and mouth. A fast rising Mississippi coincident with small contribution from the Red and Ouachita system produces high velocities in Old River. These velocities are sometimes much higher than those attained in the Atchafalaya at the same time. Conversely, a rapidly falling Mississippi following a flood crest results in low velocities in Old River with sustained high velocities in the Atchafalaya. Old River flow is also affected by appreciable flow from the Red and Ouachita system. In 1945 a large contribution from the latter system coincident with a falling Mississippi resulted in a reverse flow in Old River. This was not uncommon prior to 1945. In a

[3] U. S. Dept. of Agric., SCS, Tech. Bulletin No. 1026, September, 1950.

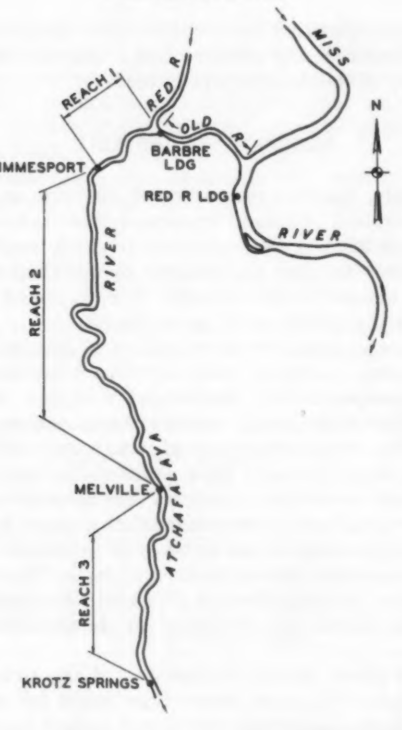

FIG. 2.—SEDIMENT STUDY REACHES

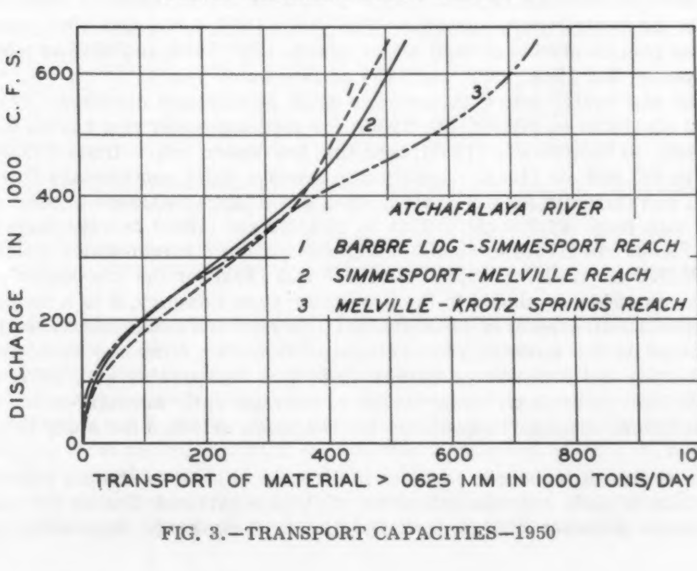

FIG. 3.—TRANSPORT CAPACITIES—1950

few instances since that date flow has been reduced to very small amounts, but there has been no recurrence of reverse flow. This is attributable to the increased flow capacity of the Atchafalaya River.

SEDIMENT TRANSPORT

As noted previously, the interrelated river reaches under discussion provide an excellent example of sediment transport capacity imbalance. To present the magnitude of imbalance it is necessary to state sediment loads quantitatively. Also, in order to show the degree of imbalance for a full range of water-type years, a number of years must be considered. This involves laborious computations if carried out in great detail.

It was not considered necessary for purposes of illustrating this imbalance to include the entire bed material load, nor was it considered necessary to make detailed load computations. Mechanical analyses of a number of bed-material samples taken in the study reach showed medium sand (0.25 to 0.50 mm) to be the dominant size, comprising approximately 35% of the total. Since any grain size group would serve to show relative transport capacities quantitatively, determinations were made only for the medium sand fraction. For analytical load determinations, it was found that a close approximation of the results of detailed computations could be attained by simple formulas requiring only knowledge of the average velocity and mean depth. These formulas, applied to velocities and depths, as measured at frequent intervals at the Torras and Simmesport discharge ranges, are the bases for the quantitative medium sand-loads presented.

An examination of mean annual discharges of the principal contributary rivers led to the adoption of three water-type years for study. These were classified according to the magnitude of average annual flow in the Mississippi River as high-, medium-, and low-water years. For each of these types three years were selected as representative of various combinations of mean annual flows in the contributary streams. The years 1950, 1945, and 1951 were selected as representative of high-water years; 1957, 1958, and 1946 as medium-water years; and 1944, 1953, and 1954 as low-water years.

In the high-water years the average daily Mississippi discharge was from 826,000 cfs (1951) to 879,000 cfs (1950); for medium-water years from 605,000 cfs (1946); to 685,000 cfs (1957); and for low-water years from 333,000 cfs (1954) to 567,000 cfs (1944). Variation in average daily contributary flow from the Red and Ouachita Rivers system, that has a large influence on flow in Old River, was from 46,000 cfs (1951) to 102,000 cfs (1945) for the high-water years; 70,000 cfs (1958) to 88,000 cfs (1957) for the medium-water years; and from 25,000 cfs (1954) to 55,000 cfs (1944 and 1953) for the low-water years.

Although 1944 was placed in the low-water year category, it is a borderline case from the standpoint of Mississippi River flow and could just as well have been placed in the medium-year category. However, from the standpoint of Red-Ouachita and Atchafalaya Rivers flow, it is comparable with 1953, which is a true low-water year. A tabulation of average daily discharges and computed sediment transport capacities for the years selected for study is shown in Table 1.

Since sediment transport capacity is directly related to stream velocity, a comparison of daily average velocities at Simmesport and Torras for the selected years is shown in Figs. 4, 5, and 6. Fig. 4 shows the high-water years

1950, 1945, and 1951. In each of these years the annual medium sand transport capacity of the Atchafalaya River exceeded that of Old River. The greatest difference was in 1945 and amounted to nearly 4 million tons. The next highest was 1950 with nearly 3 million tons, followed by 1951 with one million tons. A comparison of the velocity graphs shows why the difference was so much greater in 1945 than 1951.

The volume of water passing Natchez in 1951 was about equal to that in 1945, but the peak discharge was much higher in 1945. This is reflected in the

TABLE 1.—AVERAGE DAILY DISCHARGES AND COMPUTED SEDIMENT TRANSPORT CAPACITIES

Period	Average Daily Discharge in cfs x 10^{-3}				Medium Sand[2] Transport Capacity in Million Tons		Excess Capacity of Atch. River over Old River (Medium sand fraction in Tons)
	Mississippi River[b]	Atchafalaya R.	Old River	Ouachita and Red[c]	Old River	Atchafalaya River	
			High Water Years				
1950	879	297	195	86	5.699	8.531	2,832,000
1945	832	264	147	102	3.396	7.233	3,837,000
1951	826	256	195	46	3.925	4.955	1,030,000
			Medium Water Years				
1957	685	238	135	88	3.527(3)	-	(4)-(3) 531,000
22 Apr-3 Aug	-	-	-	-	1.234	4.058(4)	2,824,000
25 Oct-31 Dec	-	-	-	-	0.463	1.168	705,000
1958	626	233	129	70	4.094	6.192	2,098,000
1946	605	202	104	86	2.304	4.542	2,238,000
			Low Water Years				
1944	567	159	96	55	2.084	3.977	1,893,000
1953	449	152	79	55	-	-	-
1 May-31 Jul	-	-	-	-	0.274	1.839	1,565,000
1 Aug-31 Dec	-	-	-	-	0.248	0.187	-61,000
1954	333	94	63	25	1.229	1.471	242,000

[2] Bed material samples show the average medium sand fraction to be about 35%. This percentage was used in determination of transport capacities.
[b] Above Old River.
[c] Principal tributaries to the backwater area.

hydrographs shown in Fig. 4 for the Atchafalaya River, the 1945 peak being 630,000 cfs and in 1951, 425,000 cfs. Also in 1945 there were sustained high flows from the Red-Ouachita system which was a deterrent to flow through Old River. Consequently, Atchafalaya River velocities were greater for 90% of the period from January 1 to August 17th. Old River velocities were dominant for the most part only during the low-water season at which time sediment loads are relatively small.

The year 1951 was characterized by sustained periods of near-bankfull flow in the Mississippi River. In 8 of the 12 months its flow equalled or exceeded

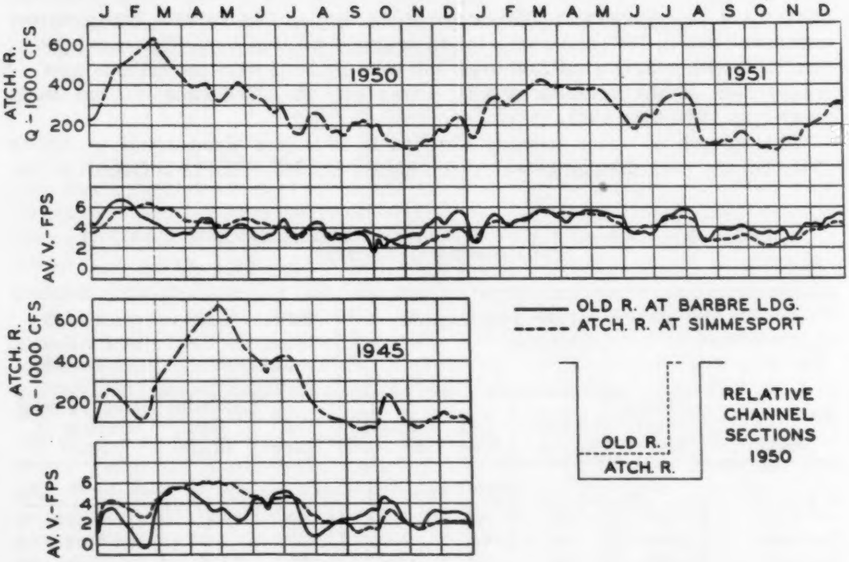

FIG. 4.—VELOCITY RELATION—HIGH WATER YEARS

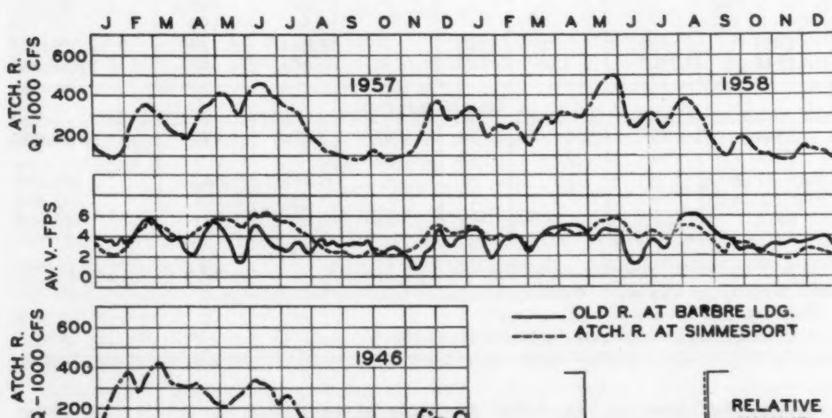

FIG. 5.—VELOCITY RELATION—MEDIUM WATER YEARS

1,000,000 cfs. Flows less than 300,000 cfs occurred in only 4 days during the year. By contrast, in the year 1945, there were only 5 months in which the flow equalled or exceeded 1,000,000 cfs, and there were 53 days when the flow was less than 300,000 cfs. In the year 1951 there were no sustained periods of appreciable flow from the Red and Ouachita system. Therefore, flow in Old River was generally steady, and its mean stream velocity compared favorable with that at Simmesport throughout most of the year. As a result, the excess sediment transport capacity of the Atchafalaya River was much less than in 1945.

The year 1950 was characterized by more than usual dominance of Old River velocity during both the spring rise and the low-water season. However, this was more than offset by greater velocities in the Atchafalaya River during most of the high-water period. The superior velocities and large volume of flow in

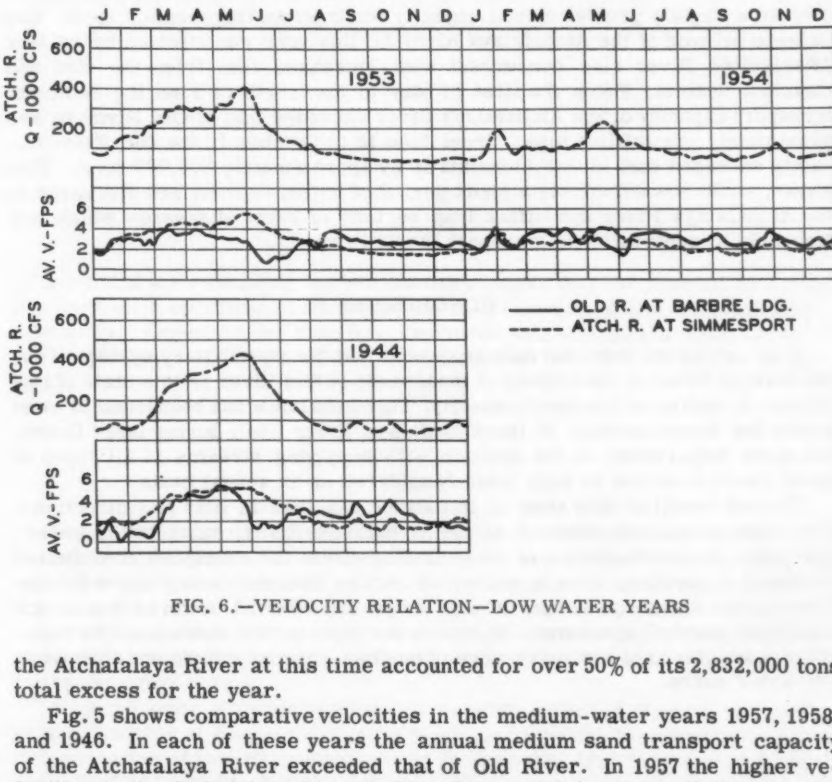

FIG. 6.—VELOCITY RELATION—LOW WATER YEARS

the Atchafalaya River at this time accounted for over 50% of its 2,832,000 tons total excess for the year.

Fig. 5 shows comparative velocities in the medium-water years 1957, 1958, and 1946. In each of these years the annual medium sand transport capacity of the Atchafalaya River exceeded that of Old River. In 1957 the higher velocities in the Atchafalaya River in May, June, and July were sufficient to permit the Atchafalaya River a sediment-transport-capacity during this period alone to exceed that for the entire year in Old River. This illustrates the potential of circumstances for great imbalance in sediment transport capacity. This year is also characterized by the abnormal situation of higher velocities in the Atchafalaya River during most of the low-water period. In both 1958 and 1946 the excess capacity of the Atchafalaya River was more than 2 million tons of the medium sand fraction.

Fig. 6 shows comparative velocities in the low-water years. As in other water-type years, the Atchafalaya River's capacity for transporting sand exceeds that of Old River in each of these cases. Here again the potential for great imbalance is illustrated. For example, in May, June, and July, 1953, the transport capacity of the Atchafalaya River was nearly 7 times that of Old River. Further, in those 3 months the transport capacity of the Atchafalaya was over 7 times that of Old River for the 5 months, August through December, a low-water period during which velocities in Old River consistently exceeded those in the Atchafalaya River.

In 1944 the excess transport capacity of the Atchafalaya was comparable to some medium-water years, and nearly twice that of high-water year 1951. Year 1954 was an exceptionally low water year. Even so, and in spite of superior velocities in Old River during most of the year, the Atchafalaya River showed a slightly greater annual capacity for transporting medium sand. The balance in favor of the Atchafalaya River for this year was effected by the May Mississippi River rise concurrent with increased flow from the Red and Ouachita system. From the first of May to the fourth of June the sediment transport capacity of the Atchafalaya River exceeded that of Old River by approximately one million tons. From June 15 to October 10 the Old River capacity exceeded that of the Atchafalaya by approximately 300,000 tons. This shows, again how effectively a short period of higher velocity and discharge in the Atchafalaya River can offset long periods of lower discharge which are favorable to superiority of transport in Old River.

CONCLUSIONS

It is conclusive from the data presented that the distributary system of the Mississippi River in the vicinity of the latitude of Old River is in a state of imbalance in sediment transport capacity. This imbalance has been found to exist within the leveed portion of the Atchafalaya River itself during large floods, and more importantly in its relation with supplying streams in all types of water years from low to high, when considered on an annual basis.

The end result of this state of imbalance can lead in only one direction—continuous annual enlargement of the Atchafalaya River regardless of water-type year. In substantiation of these findings from the standpoint of sediment transport capacities, it is a matter of record that the rating curve for the Simmesport discharge range, during the period 1951-58, lowered 2 ft at approximate bankfull discharge. In this seven-year period there were no high-water years; three of the years were of medium-year magnitude and four were low-water years.

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DRAG AND LIFT ON SPHERES WITHIN CYLINDRICAL TUBES

By Donald F. Young[1]

SYNOPSIS

A method for evaluating the lift and drag forces that are exerted by a moving fluid on a spherical particle resting on the bottom of a cylindrical tube is described. Experimental data are presented to show the variation in the lift and drag force with the particle to pipe-diameter ratio and the Reynolds number.

INTRODUCTION

One of the most important aspects of the general problem of the flow of suspensions is the study of the conditions necessary for maintenance of the solid particles in suspension. This specific problem is, in turn, related to the evaluation of the fluid forces that act on the individual particles. The resultant dynamic fluid force acting on a particle can be resolved into two components, a drag force parallel to the direction of flow, and a lift force normal to the direction of flow.

Although some attention has been given to the effect of the drag component on the behavior of suspensions, little work has been done in connection with the determination of the lift component. It is probably true that in certain problems the lift force has little significance, however, for the study of incipient motion of particles resting on the bottom of channels or pipes the lift force

Note.—Discussion open until November 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 6, June, 1960.

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may be of primary importance. S. Leliavsky has summarized[2] the work that has been done in this area and points out that it is "an almost unexplored aspect of the problem." The purpose of the investigation reported on herein was to develop a method for evaluating the lift and drag forces that act on a single spherical particle located on the bottom of a tube, and to study the parameters that affect the magnitude of these forces.

MEASUREMENT OF LIFT AND DRAG FORCE

The free-body diagram for a spherical particle in contact with a fixed boundary inclined at an angle β is shown in Fig. 1. The resultant dynamic fluid force has been resolved into a lift component L and a drag component P. It should be noted that, in general the resultant fluid force does not pass through the center of gravity of the particle. C. M. White has investigated[3] the location of the drag component in some detail. The weight of the particle minus

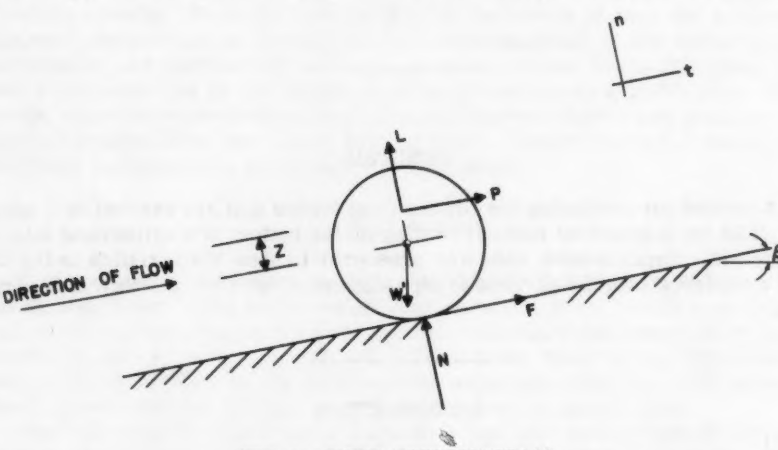

FIG. 1.—FREE-BODY DIAGRAM

the buoyant force is denoted by W' (W' > 0) and the forces F and N are the components of the boundary reaction on the particle.

Equilibrium equations for the particle are

$$L = W' \cos \beta - N \quad \text{.......................(1)}$$

in the n - direction,

$$P = W' \sin \beta - F \quad \text{.......................(2)}$$

in the t - direction, and

$$P\,Z = \frac{d}{2}\,F \quad \text{.......................(3)}$$

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2 "An Introduction to Fluivial Hydraulics," by S. Leliavsky, Constable and Co., 1955, p. 64.

3 "The Equilibrium of Grains on the Bed of a Stream," by C. M. White, Proceedings, Royal Soc. of London, A174, 1940, p. 322.

for moments with respect to an axis through the center of gravity of the particle of diameter d.

By varying the rate of flow past the particle, the drag component P can be changed and the particle made to move up or down the plane with a constant velocity or to remain stationary. As the angle β is increased, a greater velocity is required to hold the particle stationary or move it up the plane. However, as β increases, the component of the apparent weight normal to the pipe wall W'_n that opposes the lift force decreases. Therefore at some critical angle it should be possible to have the lift force equal W'_n so that any slight increase in the value of L will cause the particle to rise off the bottom of the pipe. Since W'_n can be readily measured, the lift force can be determined for this condition of impending motion. Also for this condition of impending motion the drag force is equal to $W' \sin \beta$ since $F = 0$. Thus both the lift and the drag force can be determined. This method of determining these forces has the advantage that no attachments need be made to the particle to hold it in position.

However, it should be noted that as motion impends in the direction of the lift force both the normal force N and the frictional force F approach zero. Therefore in order that no rotation take place, Eq. 3 indicates that the product P Z must approach zero. Since $P \neq 0$ this implies that either $Z \equiv 0$ or that Z approaches zero as the lift force approaches W'_n. It is thus questionable whether impending motion in the direction of the lift force will occur without rotation. However, actual observations indicated that a particle can be made to rise off the bottom without rotation indicating that the necessary conditions were satisfied, at least for the tests run in this investigation.

EXPERIMENTAL PROCEDURES

The experimental set-up used to measure the lift and drag forces by the procedure outlined was quite simple. It consisted primarily of a constant-head tank connected to a glass tube which could be inclined at various angles. The angles could be measured to within 1.0°. Parallel needle valves were used to control the rate of flow through the system. The fluid temperature remained relatively constant for all tests, and the fluid used in all tests was water. A schematic diagram of the equipment is shown in Fig. 2.

It was expected that for turbulent flow in the pipe the motion of the particle would be erratic and measurements difficult to obtain. For this reason it was desired to run all tests under laminar-flow conditions in the pipe. Therefore a particle of low specific weight was needed so that the desired conditions could be obtained with relatively low velocities. To meet this requirement nylon spheres with a specific weight of 1.1923 g per cu cm were used. The test facility was arranged so that various sizes of glass tubing could be used. In order that the flow at the measurement station be fully developed laminar flow, this station was located at a distance of approximately 4 ft from the entrance and 2 ft from the discharge end of the tube. A commonly used equation for estimating the transition length x is

$$x = 0.26 \, r \, R \quad \text{(4)}$$

in which r is the tube radius and R is the Reynolds number based on the tube radius.[4] The maximum transition length that occurred during the tests, as

[4] "Applied Hydro- and Aeromechanics," by L. Prandtl and O. G. Tietjens, Dover Publications, Inc., New York, 1957, p. 22.

estimated from Eq. 4, was 3.3 ft. It was therefore assumed that for all the tests the flow was fully developed at the measurement station. Table 1 summarizes the combination of particle and tube sizes used in the experimental investigation.

For a given test the angle β was fixed and the motion of the particle in the tube was controlled by the rate of flow from the constant-head tank. As expected, at some critical angle the particle could be made to rise off the bottom of the tube with essentially no motion parallel to the longitudinal axis of the tube. The particle would lift off the bottom, then the lift force would apparently decrease, and the particle would drop back so that there was actually a "hopping" motion.

The procedure followed in each test was to adjust the angle and rate of flow until the particle had impending motion normal to the pipe wall and to record the data for this condition. The rate of flow in the tube was obtained by collecting the fluid as it discharged from the tube and determining the time required to collect a given volume.

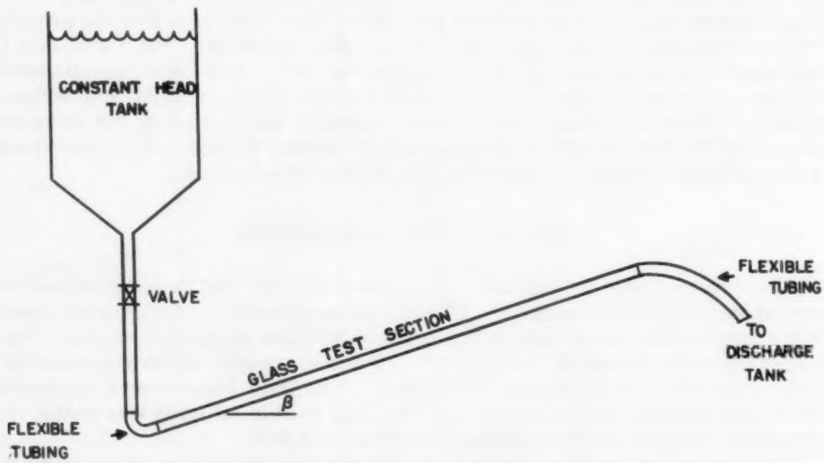

FIG. 2.—EXPERIMENTAL APPARATUS

One disadvantage to this method of measuring the fluid forces acting on a particle is that a certain amount of personal judgment is involved in establishing the condition of impending motion of the particle. In order to have some index to the reliability of the resulting data, each test with a particular combination of particle and pipe size was repeated at least seven times and the standard deviation of the lift and drag coefficients was evaluated. The standard deviation from the mean values of the lift and drag coefficients was less than 3% of the mean values for all tests. As indicated in Table 1 the range of values for the (d/D)-ratio was from 0.282 to 0.635. The Reynolds number range, based on the pipe diameter and mean velocity, was 360 to 1,117.

TABLE 1.—SUMMARY OF EXPERIMENTAL RESULTS

$\frac{d}{D}$	d, in cm	D, in cm	R_d	R_D	C_D	C_L
0.282	0.321	1.135	220	780	1.50	0.61
0.345	0.480	1.389	385	1,116	1.61	0.73
0.353	0.401	1.135	284	805	1.73	0.78
0.399	0.321	0.803	166	416	2.58	1.18
0.403	0.559	1.389	450	1,117	1.86	0.87
0.423	0.480	1.135	330	780	2.17	1.04
0.459	0.638	1.389	495	1,078	2.26	1.10
0.493	0.559	1.135	380	771	2.58	1.27
0.500	0.401	0.803	204	408	3.30	1.62
0.519	0.721	1.389	538	1,037	2.76	1.36
0.562	0.638	1.135	402	715	3.39	1.73
0.598	0.480	0.803	215	360	5.02	2.58
0.635	0.721	1.135	411	647	4.67	2.43

ANALYSIS OF EXPERIMENTAL DATA

It was assumed that the variables influencing the lift force are the mean velocity V, fluid density ρ, fluid viscosity μ, particle diameter d, and pipe diameter D, so that

$$L = \phi\ (V, \rho, \mu, d, D) \quad \text{(5)}$$

Eq. 5 can be written in terms of three dimensionless parameters as

$$\frac{L}{\rho\ A \frac{V^2}{2}} = \phi_1 \left(\frac{\rho\ D\ V}{\mu}, \frac{d}{D}\right) \quad \text{(6)}$$

where A is the cross-sectional area of the particle and the constant 2 has been arbitrarily introduced to put the equation in a more conventional form. Therefore the expression for the lift force is

$$L = C_L\ \rho\ A \frac{V^2}{2} \quad \text{(7)}$$

where the lift coefficient C_L is given by

$$C_L = \phi_1 \left(R_D, \frac{d}{D}\right) \quad \text{(8)}$$

The Reynolds number R_D is based on the pipe diameter. Of course a Reynolds number based on the particle diameter could have been used. However it was felt that the velocity distribution preceding the particle, which is a function of the pipe Reynolds number, would have considerable influence on the lift and drag coefficients and thus R_D was used. However the results can readily be interpreted in terms of the Reynolds number R_d based on particle diameter since

$$R_d = R_D \frac{d}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Similarly the drag force can be expressed as

$$P = C_D \, \rho \, A \frac{V^2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

where

$$C_D = \phi_2 \left(R_D, \frac{d}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Since the lift and drag coefficients can be evaluated from the experimental data, the problem reduces to one of determining the unknown functions ϕ_1 and ϕ_2. Table 1 contains all the data and Fig. 3 is a plot of the experimentally determined values of C_L and C_D versus R_D for the various diameter ratios. Due to the limited number of particle and tube sizes available and the manner in which the data were obtained it was not possible to hold either d/D or R_D constant for a series of tests. It was therefore difficult to correlate the data. However an examination of Fig. 3 suggested that both the lift and drag coefficients were inversely proportional to $R_D{}^{1/3}$. It was further assumed that the equations for the lift and drag coefficients could be written as

$$C_L = \frac{1}{R_D{}^{1/3}} \; f_1\left(\frac{d}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

and

$$C_D = \frac{1}{R_D{}^{1/3}} \; f_2\left(\frac{d}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

It is apparent that if these assumed forms are correct then a plot of $C_L \; R_D{}^{1/3}$ and $C_D \; R_D{}^{1/3}$ versus d/D should correlate all the data, and these plots were used to indicate the validity of Eqs. 12 and 13. It was found that for the range of variables covered in the investigation the data do appear to correlate satisfactorily as shown in Figs. 4 and 5. The empirical equations for the drag and lift coefficients which fit the data shown in these figures are

$$C_D = \frac{1}{R_D{}^{1/3}} \left[94.5 \left(\frac{d}{D}\right)^{2.57} + 10.0 \right] \quad \ldots\ldots\ldots\ldots (14)$$

and

$$C_L = \frac{1}{R_D{}^{1/3}} \left[50.5 \left(\frac{d}{D}\right)^{2.28} + 2.50 \right] \quad \ldots\ldots\ldots\ldots (15)$$

It should be emphasized that although the relatively simple form for correlating the data appears to be reasonably satisfactory, it is not likely that it will

be valid over the entire range of Reynolds numbers that is commonly of interest. In this investigation the range of Reynolds numbers based on particle diameter R_d was 166 to 538. It is well known from conventional drag-coefficient tests for spheres in an essentially unbounded fluid that this range of Reynolds numbers represents a transition region and that a simple valid expression for the drag coefficient in this range cannot be extended into the regions where viscous forces are predominant or into regions where viscous forces are negligible.

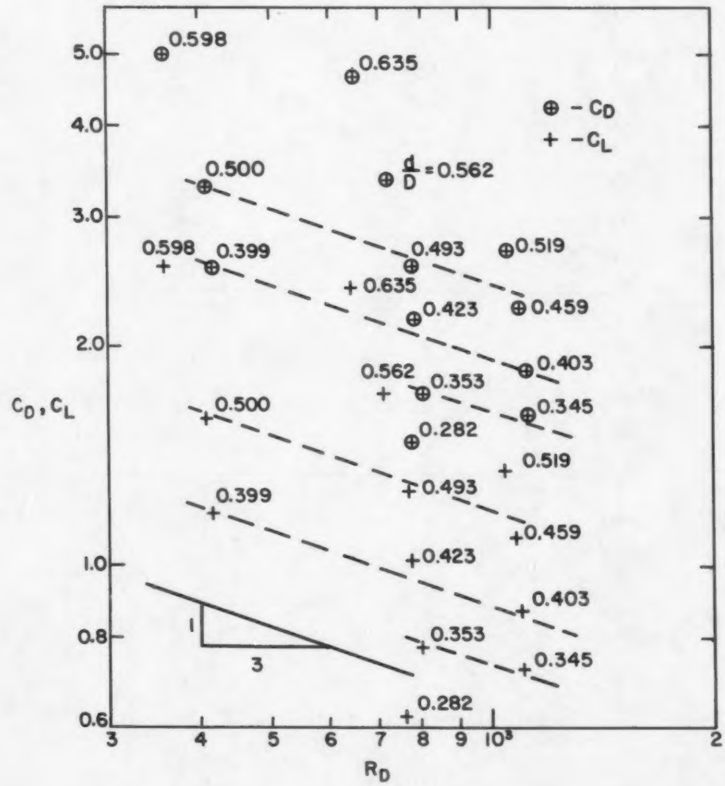

FIG. 3.—VARIATION OF COEFFICIENTS WITH REYNOLDS NUMBER

Figs. 4 and 5 indicate that for a given Reynolds number both the lift and drag forces increase with increasing values of d/D and for a given value of d/D both the lift and drag forces decrease as the Reynolds number increases. These results for the drag coefficients are in qualitative agreement with those obtained[5] by J. S. McNown and J. T. Newlin in their study of the drag force acting on spheres located in the center of a cylindrical tube. Data obtained

[5] "Drag of Spheres within Cylindrical Boundaries," by J. S. McNown and J. T. Newlin, Proceedings, First U. S. Natl. Congress of Applied Mechanics, 1951, p. 801.

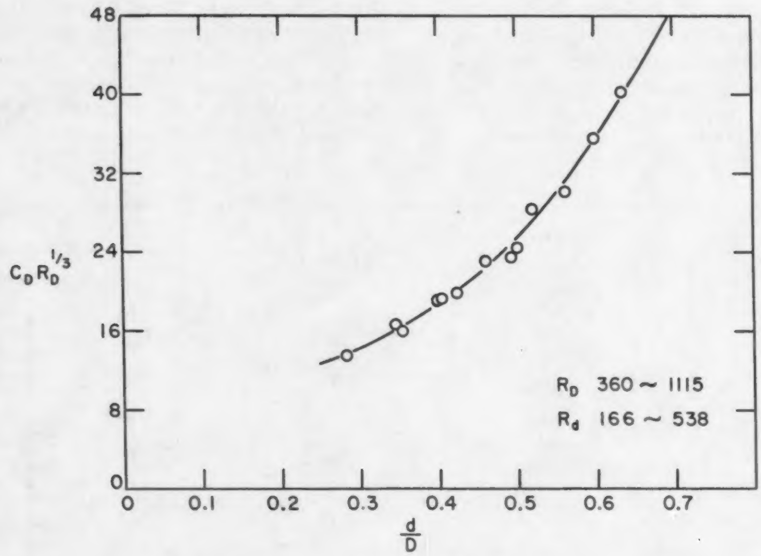

FIG. 4.—CORRELATION OF DRAG DATA

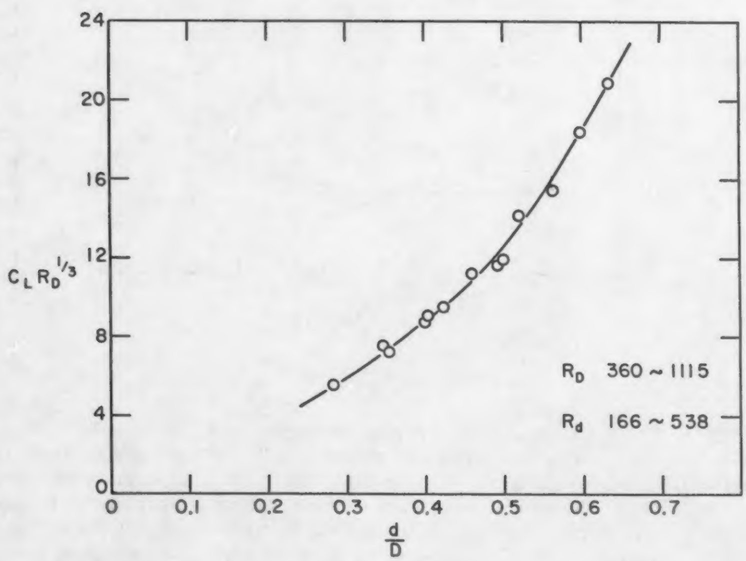

FIG. 5.—CORRELATION OF LIFT DATA

from curves given by McNown and Newlin are plotted in Figs. 6 and 7. It is noted that the drag coefficients for spheres located near the wall of a tube in which the velocity of approach has a parabolic distribution are considerably higher than those for spheres located in the center of the tube in which the velocity of approach is uniform. J. Happel and B. J. Byrne derived[6] analytical expressions for the drag force acting on a sphere in the Stokes range placed in the center of a cylindrical tube for both the case with a uniform approach velocity and with an approach velocity with a parabolic distribution. They found that the drag coefficients for the case with the parabolic distribution were roughly twice those for the case with the uniform distribution if the characteristic velocity is taken as the mean velocity. This difference in drag coefficients is of the same order of magnitude as those indicated in Figs. 6 and 7.

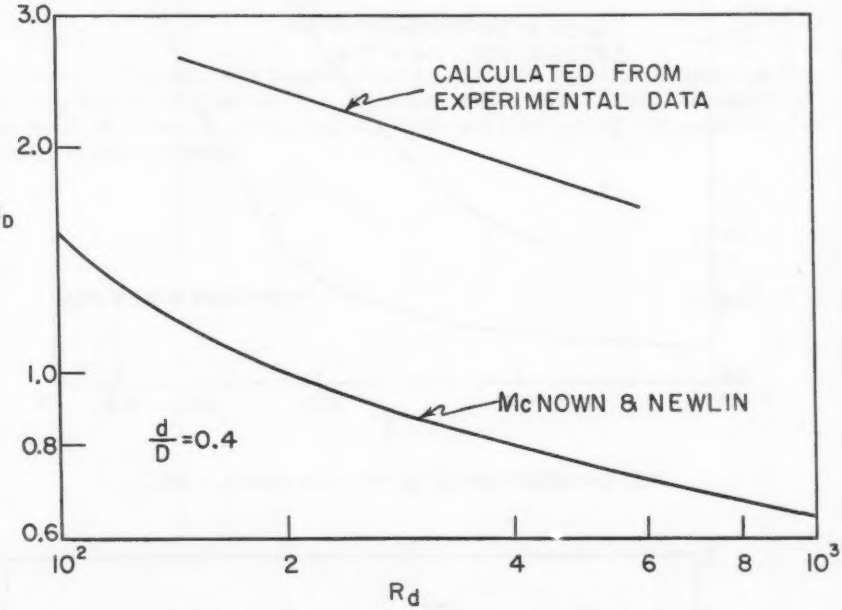

FIG. 6.—VARIATION OF C_D WITH R_d FOR D/d = 0.4

From Figs. 4 and 5 it is apparent that the ratio of the drag to the lift force is a function of only d/D. The plot of this ratio versus d/D is shown in Fig. 8.

CONCLUSIONS

In this study a method was developed to determine the lift and drag forces acting on a spherical particle resting on the bottom of a circular tube. A number of tests was performed to confirm the feasibility of using this method and to determine the values of the lift and drag forces. It is concluded that the

6 "Motion of a Sphere and Fluid in a Cylindrical Tube," by J. Happel and B. J. Bryne, Industrial and Engineering Chemistry, Vol. 46, 1954, p. 1181.

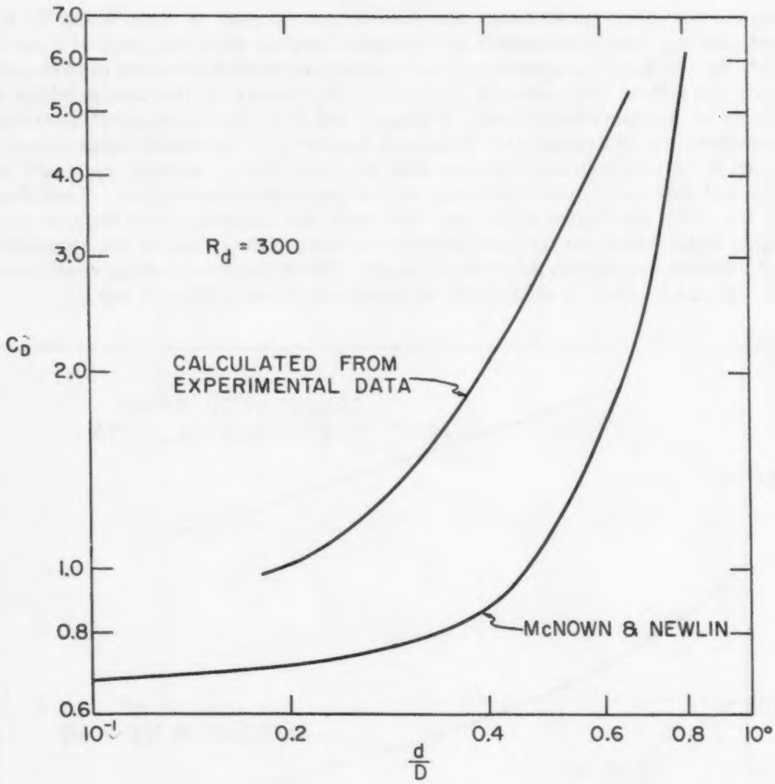

FIG. 7.—VARIATION OF C_D WITH D/d FOR R_d = 300

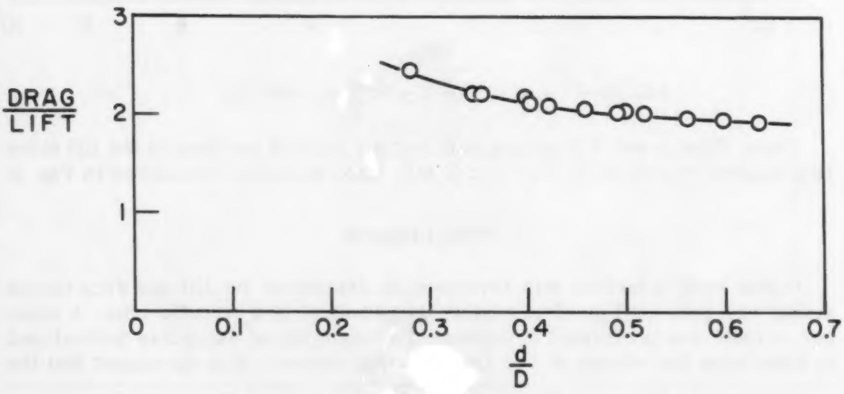

FIG. 8.—VARIATION OF DRAG TO LIFT RATIO WITH d/D

proposed technique can be used to measure these forces accurately. In addition, Fig. 8 shows that the lift force is of the same order of magnitude as the drag force and that the ratio of drag to lift, although a function of the d/D-ratio, is approximately equal to 2.

The range of values of the parameters studied in this investigation is limited, but the same type of equipment and technique can be used to extend the range with the proper choice of fluids and particles. Probably the most important result of this study is the establishment of the fact that for spheres near the wall of a tube a lift force with a magnitude comparable to that of the drag force does actually exist. It is thus apparent that the lift force should not be overlooked in studies related to the incipient motion of particles resting on stream beds or pipe walls.

ACKNOWLEDGMENTS

This investigation was conducted as a part of the research program of the Nuclear Engineering Group 1 of the Ames Laboratory. The assistance of James R. Melcher in conducting the tests and performing computations is gratefully acknowledged.

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MODELS PRIMARILY DEPENDENT ON THE REYNOLDS NUMBER

By W. P. Simmons, Jr.,[1] M. ASCE

SYNOPSIS

The design, construction, and operation of models of closed-conduit fluid systems, and interpretation of test data are discussed. Viscous and inertia forces in the flowing fluids of such models, and hence Reynolds number, are predominant factors. General rules for model size, construction, instrumentation, operating Reynolds number ranges, and data analysis are given.

INTRODUCTION

Hydraulic studies of models of closed-conduit flow systems provide a reliable means for obtaining useful and accurate design data. However, there are pitfalls in the use of models that must be recognized and avoided if the necessary accuracy and reliability are to be obtained. Thus, a knowledge of the similitude relationships between models and their full-sized counterparts is essential. An excellent discussion of these similitude factors has been presented by H. K. Liu and M. L. Albertson,[2] and by H. Rouse;[3] therefore, a full account of the details will not be given here. It will be in order, however, to define what is meant by closed-conduit flow and to state briefly the fundamental model relationships that must be observed.

Note.—Discussion open until November 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 6, June, 1960.

1 Hydr. Engr., Div. of Engrg. Labs., Bur. of Reclamation, Denver, Colo.

2 "Significance and Application of Froude and Reynolds Numbers as Criteria for Similitude," by H. K. Liu and M. L. Albertson, presneted at the June 1959 ASCE Convention in Fort Collins, Colo.

3 "Engineering Hydraulics," by H. Rouse, John Wiley and Sons, Inc., New York, 1950, Chapter 2.

Closed-conduit flow is defined as flow in an enclosed system where no free water surface exists, or flow around a deeply submerged body where the effect of a free water surface is negligible. Typical examples of structures involving closed-conduit flow include piping systems, pressure tunnels, control gates, pumps, and turbines.

In studies of these systems, gravitational forces are not a factor affecting the flow because the flow direction always will be down the slope of the energy grade line and independent of the slope of the pipe itself. Surface wave motion and surface tension are nonexistent because there is no free surface, and, therefore, are not a consideration. Elastic compression of the fluid, which in the case of gases can be significant, is usually negligible with liquid flow. Thus, of all the major forces and fluid characteristics affecting fluid flow, viscosity remains as the predominant factor in the usual closed conduit, hydraulic flow problem.

The Reynolds number, $\frac{\rho V D}{\mu}$ or $\frac{V D}{\nu}$ is the ratio of the fluid inertia forces to the fluid viscous forces. The number is dimensionless, and ρ is the fluid density, V denotes the flow velocity, D is a characteristic dimension (usually the diameter in pipes), and μ and ν are the absolute and kinematic fluid viscosities, respectively. Values for ν for fluids at normally encountered temperatures are given in Fig. 1.

Usually the Reynolds number of a prototype structure is very large. Duplicating this number on scale models is impractical, particularly if the same fluid is used on both structures, for, as the size decreases, velocities must increase proportionately. With high initial prototype velocities, the problem rapidly would seem to become impossible.

Study of the Reynolds number equation shows that the effects of viscosity are particularly important when flow velocities and, hence, inertia forces, are low. But when flow velocities are high, inertial forces assume such great significance that viscosity becomes relatively minor. This is a basic factor that makes it possible to achieve reasonable similarity between a model and its larger-sized prototype. By making the model large enough, or by using velocities high enough, or by both, inertial forces can be made to predominate over viscous forces and a reasonable similarity with the prototype will exist. The other requirement is that the model be large enough to insure flow in the fully turbulent range whenever full turbulence occurs in the prototype. Thus, the procedure followed in making usual model studies of elements of closed-conduit hydraulic systems is to build reasonably large, very accurate models, and to operate them with flow velocities equal to or higher than scaled velocities. Full turbulence is generally encountered on prototype structures, and the discussion in this paper will be limited to these conditions.

MODEL DESIGN

A first consideration in designing a model is the selection of the testing fluid. Water, the most generally available and perhaps most satisfactory liquid, is commonly used. Water also is usually the fluid used in prototype structures and naturally associated with model studies. In many types of closed-conduit testing, air can be used as the testing fluid with a real savings in time and costs.[4] For this reason, low-velocity air tests should be considered in the

4 "Model Tests Using Low Velocity Air," by J. W. Ball, Transactions, ASCE, 1952.

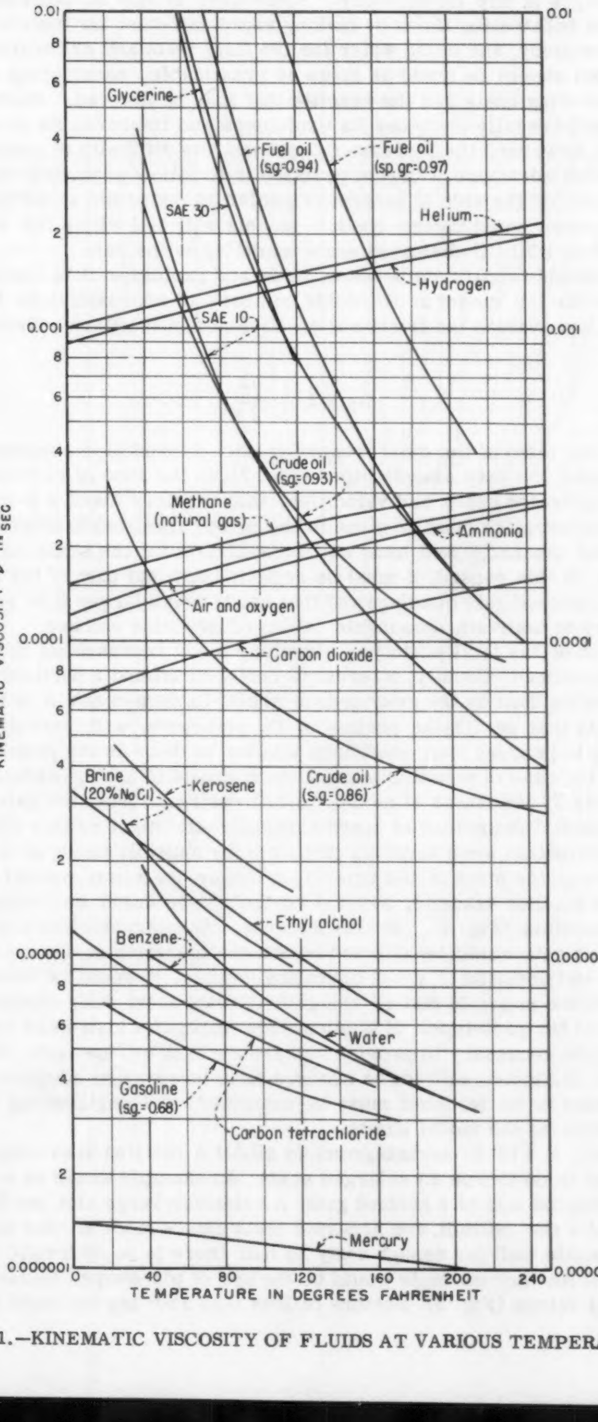

FIG. 1.—KINEMATIC VISCOSITY OF FLUIDS AT VARIOUS TEMPERATURES

planning stages of any model study. Sometimes it may be desirable to take advantage of both testing fluids by making rapid, low-cost air tests in the early stages of the study, and using water for the final tests and calibrations.

The model should be made as large as practicable, considering construction and operating costs and the benefits that may be derived. Increasing the size of a model usually enhances its usefulness and improves its accuracy. At some point, however, the increase in cost and the difficulty of operation will offset any size advantage. Current practice is to follow precedent when available and to err on the side of largeness insofar as space and water supply permit. In general, most closed-conduit models will fall within the scale ratio range of 1:5 to 1:30. Undistorted scale modeling is the rule.

For reasonable similitude between model and prototype, it is usually necessary to operate the model at Reynolds numbers of approximately 1×10^6 or more. At these values the frictional coefficient f in the Darcy-Weisback formula

$$h_1 = f \frac{1}{d} \frac{V^2}{2g} \qquad (1)$$

is constant for many of the most frequently encountered pipe roughnesses, and nearly constant for very smooth pipes (Fig. 2). In the case of valve or conduit models using scaled heads and velocities, this requires about a 6-in. pipe diameter. Consideration must be given to the pump capacities to determine whether adequate discharge and head will be available for the scale ratios being considered. In this regard, it must be remembered that part of the tests may be made at reduced gate openings and that heads much larger than scaled may be necessary to maintain reasonable values of Reynolds number.

The extent of the total prototype structure to be represented in the model must be determined. Usually, interest is centered around a particular part of a system, rather than on the system as a whole. In such cases, it is necessary to model only that particular portion of the structure, with enough inlet and outlet piping to produce flow conditions similar to those in the prototype. For example, in the case of a control gate at the midpoint of a long conduit in a dam approximately 20 diameters of model conduit upstream from the gate, the gate itself, and about 5 diameters of conduit downstream might suffice (Fig. 3). If the flow distribution were severely distorted by multiple bends or a partially opened valve at the start of the conduit, a longer upstream conduit would be needed. As another example, several control gates might be connected to a branching manifold (Fig. 4). In this instance, the characteristics of the flow reaching each gate would be affected by the configuration of the manifold and the number and location of gates operating. Thus, it would be necessary to model the entire manifold and all the gates to represent fully conditions that could occur in the prototype. If studies were made of a high-head conduit inlet, it might be necessary to provide a pressure tank to represent the reservoir (Fig. 5). In studies with lower heads, a head box may be adequate (Fig. 6). Each structure to be modeled must be examined, and engineering judgment used, to determine the model limits.

Frequently, it will be advantageous to model a critical area only within a structure and to do this on an enlarged scale. An example would be a gate slot for restraining the leaf of a control gate. A relatively large slot can be formed in the wall of a test conduit, and accurate tests can be made if care is taken to have the opposite wall far enough away so that there is no hydraulic interference (Fig. 7). Another example would be the use of pie-shaped sectors of circular control valves (Fig. 8). Sectors of less than 180° are the most common.

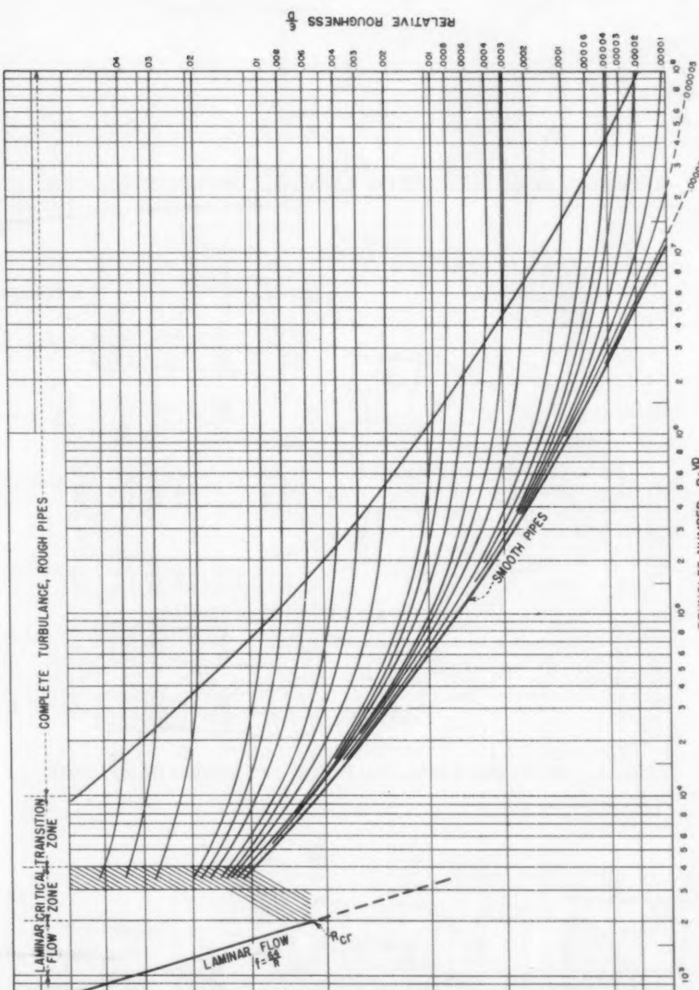

FIG. 2.—MOODY DIAGRAM SHOWING RELATION OF FRICTION FACTORS TO REYNOLDS NUMBER

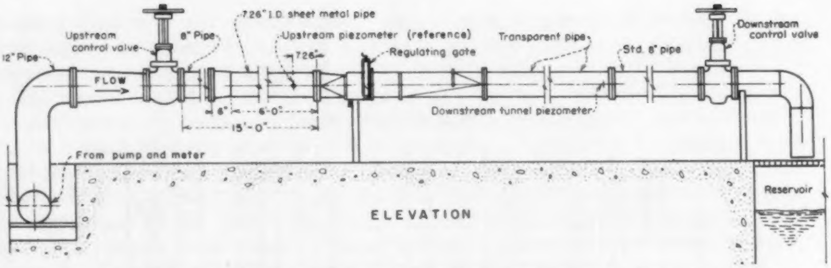

FIG. 3.—GENERAL ARRANGEMENT OF TYPICAL CONTROL STRUCTURE MODEL

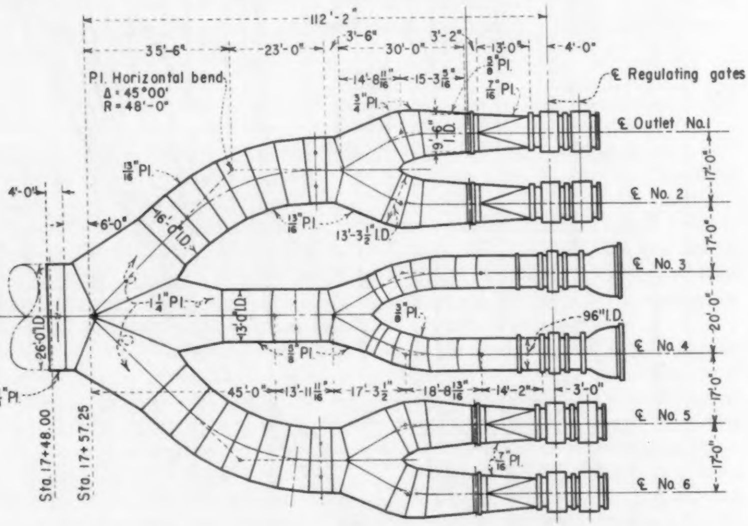

FIG. 4.—BRANCHING MANIFOLD FOR PALISADES DAM, IDAHO

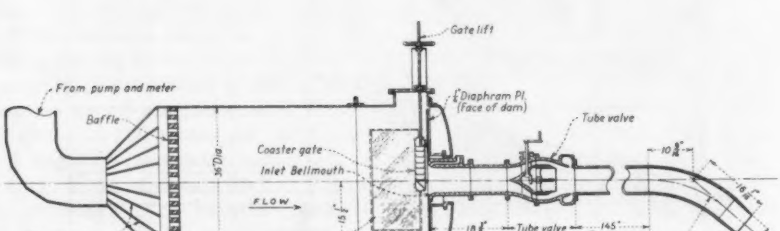

FIG. 5.—PRESSURE TANK FOR HIGH-HEAD INLET TESTS

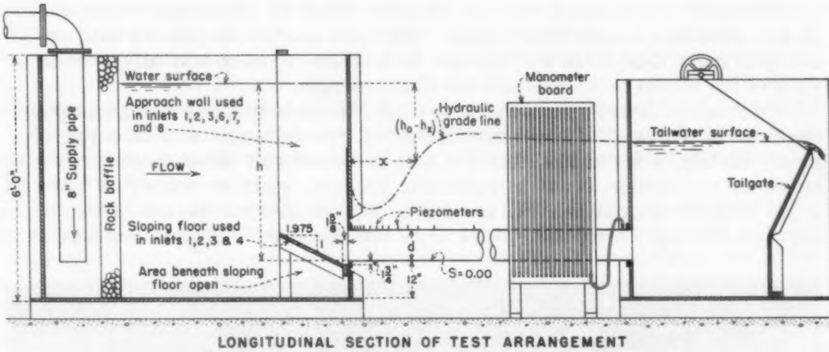

FIG. 6.—HEAD BOX FOR LOW-HEAD INLET TESTS

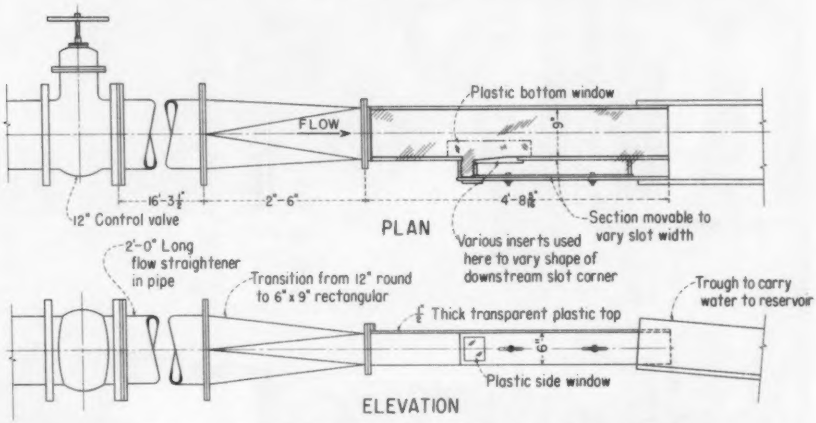

FIG. 7.—SECTIONAL TEST FACILITY FOR GATE SLOTS

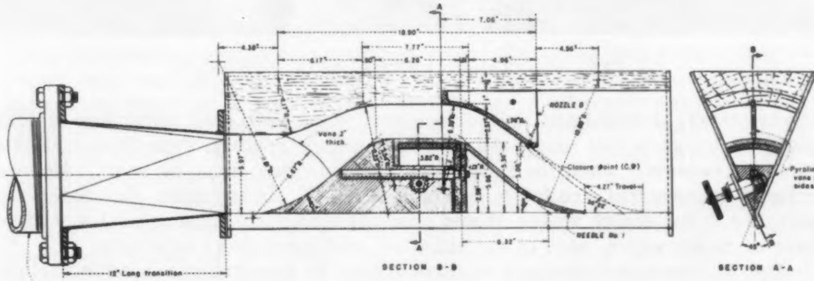

FIG. 8.—ONE EIGHTH SECTIONAL MODEL OF NEEDLE VALVE

Time and unnecessary expense may be saved by selecting the scale ratio to accommodate standard sized pipes and other model components that are already on hand. Odd scale ratios, such as 1:17.35 will in no way affect the accuracy of the model or complicate the data analysis.

Materials ordinarily used in water models include bronze castings, plate and sheet brass, galvanized sheet steel, plastics, concrete, wax, and modeling clay. In air models, wood and plaster are also satisfactory. When moderately high heads and velocities occur in hydraulic models, rigid construction must be used. Models circular in cross section, such as needle valves, use bronze castings to advantage. When the shapes are rectangular in cross section, machined

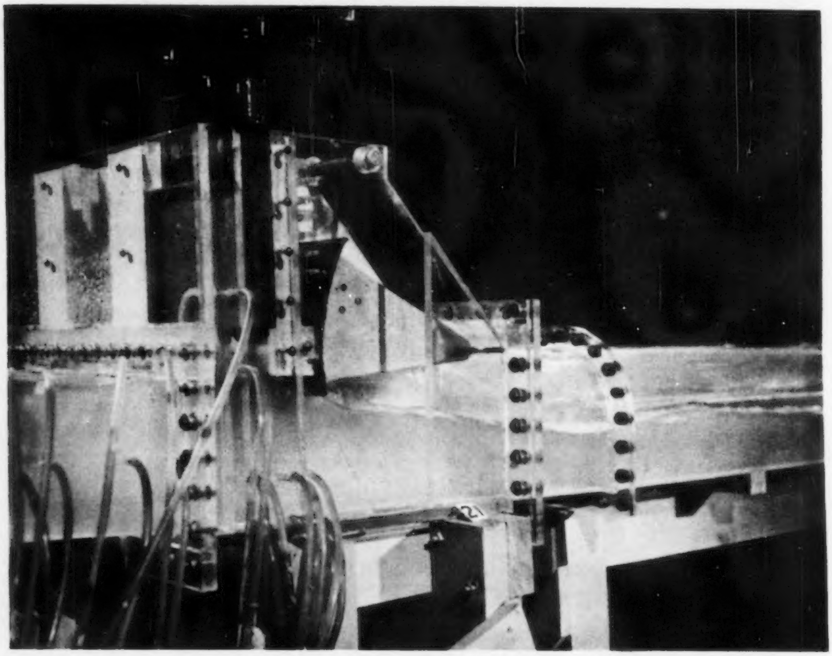

FIG. 9.—TRANSPARENT PLASTIC MODEL

brass plates, doweled and bolted together, work well. An advantage of this build-up design is that model changes can be made readily. Transparent plastic is used frequently when the flow within the structure must be seen. Models entirely transparent are fairly common (Fig. 9). In any case, the materials selected in the design stages should combine durability, dimensional stability, ease of construction, ease of modification, and corrosion resistance.

The inclusion of measuring equipment such as piezometers is an important design consideration. Often the requirement of getting the instruments in place within the model dictates the type of construction used. Piezometers are relatively cheap when included in the initial model construction, and should be used generously to offset oversights in defining critical areas. Thought should

therefore be given to the necessary instrumentation and how it will be included in the model so the desired information will be obtained.

The testing fluid must be supplied to the model in a manner representative of prototype conditions. Appropriate conduit length must be provided between any supply system bends or controls, and the model itself, so that the flow distribution will be developed to the proper extent by the time it reaches the test section. Straightening vanes are useful in reducing the tendency for spiralling flow within the pipes.

The testing fluid may be supplied directly from pumps, or from surge or constant head tanks. Ideally, it would be desirable to have each model supplied through a constant-head tank, but in many cases this is unnecessary and excellent results are obtained with the flow supplied directly from pumps. The characteristics of the pumps being used have a considerable bearing on whether or not a constant-head tank is necessary. If the pumps are used directly, it is best to run them at appreciable heads and to throttle the flow to the model. In this manner, better flow stability is obtained.

The waste from hydraulic models must either be returned to laboratory reservoirs or to the rivers or lakes from which it was drawn. This may be done in the most convenient manner and has little effect on the model operation if proper outlet conditions are maintained at the model test section. Discharges from air models usually present no problems.

Accurate and complete drawings are essential for expediting model construction and preventing time consuming, expensive errors. Drawings should contain sufficient information to enable craftsmen to build a structure that conforms to the designers' specifications. Special drawings of details are often necessary. All information for the installation of instruments such as piezometers, pressure cells, velocity measuring devices, etc., should be included, thereby obviating extensive changes after the model is completed. In fortunate cases where craftsmen are experienced in hydraulic model construction, many structural details may be omitted and much of the construction method left to the shop foreman.

MODEL CONSTRUCTION

Extreme construction accuracy is essential for obtaining good results from any hydraulic or air model. Care must be taken that the materials, methods, and craftsmen are satisfactory and that tolerance checks are made frequently. Greatest accuracy should be maintained where abrupt changes in flow direction occur, and where flow velocities are high or pressures are low. The stage of the testing program affects the type and accuracy of construction. In early tests, during which many schemes may be tried to determine feasibility, construction need not be as careful as in the final stages, when data are used to describe prototype operating characteristics.

In hydraulic models of high head gates and valves, relatively thick metal or plastic sections are used and all important flow surfaces are machined. Thus, close tolerances and accurate component alinement can be obtained. In air models, wood, sheet metal, wax, and lighter plastic sections are permissible because the internal forces and the weight that the models sustain are small. The need for accuracy remains unchanged. For most water models made by the Bureau of Reclamation, United States Department of the Interior (USBR), transitions from one shape or area to another, manifolds, bends, etc., are

formed from 16 to 22 gage sheet metal, or 1/10 in. to 1/4 in. thick, hot-worked sheet plastic. In cases where flow conditions within the system are to be observed, the transparent plastic sections are ideal. They are usually comparatively expensive because wooden patterns must often be made first, and the heated plastic formed over them, cooled, trimmed, joined, and flanged into final condition. The handwork increases costs, and the advantage of visual inspection must be balanced against this extra expense.

The inclusion of piezometers often complicates the model construction and may restrict the choice of materials. In the case of castings or heavy metal sections, piezometer holes may be drilled through the bodies and connecting taps can be threaded or soldered into the exterior surfaces. When a number of piezometers are grouped within a limited area, it may be necessary to use devious passages to prevent one piezometer lead from interfering with another. In sheet metal construction, metal tubing is inserted through the metal walls, soldered in place with the tubing normal to the wall, and then smoothed flush with the flow surface. All burrs must be removed from the interior of the piezometer opening.[5] When concrete, plaster, wood, or wax is used for forming intricate flow surfaces, it is desirable to embed in the material metal tubes that extend to the surface, so that the piezometers will have durable, smooth, square-edged openings. When materials are suitable and superior craftsmanship is available so that chipping or damaging do not occur, holes can be drilled directly into the material. Usually the piezometers should be 1/16 in. or larger in inside diameter and should extend at least 2 diameters from the opening without change in cross section or alinement. Piezometers smaller than 1/16-in. diameter may be used, and may be desirable on curving surfaces or where many are crowded into a small area, but they are prone to becoming plugged with small bits of material, and tend to induce damping in water models that reduces the sensitivity of measurement. The smaller piezometers work well in air models.

Flexible tubing about 1/4-in. inside diameter is usually used to connect the piezometer taps to the pressure measuring instruments, usually water manometers. Rubber tubing has been used extensively, but is relatively short-lived and has an opaque quality that prohibits observing interior conditions. Recently, flexible, transparent plastic tubings have become available and are highly recommended. An advantage of the transparent tubing is that entrapped air bubbles can be detected easily and removed from the line.

When measurements are to be made of rapidly fluctuating pressures, care must be taken in matching piezometer diameter, pressure lead diameter, pressure lead length, the pressure cell itself, and the recorder.[6,7] Failure to do this may result in obtaining records that reflect serious distortions of the actual pressure history. Deductions made in all innocence and confidence from such records would, of course, be erroneous and misleading. In general, it should be remembered that the pressure leads should be as short as possible, rigid, and of at least 1 1/6-in. inside diameter. All entrapped air and gas bubbles should be bled from the system, and the temperature of the cell should not vary appreciably during the calibration and tests.

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5 "Piezometer Investigation," by C. M. Allen and L. J. Hooper, Transactions, ASME, Hyd 54-1.

6 "A Note on the Evaluation of Design of Transducer for the Measurement of Dynamic Pressures in Liquid Systems," by J. R. Barton, Instrument Notes, Statham Labs., No. 27, June, 1954.

7 "Attenuation of Oscillatory Pressures in Instrument Lines," by A. S. Iberall, Research Paper RP 2115, Natl. Bur. of Standards, Vol. 45, July, 1950.

Provisions should be made for installing any other measuring equipment needed on the model. For instance, it may be desirable to make velocity traverses across the conduits, and pitot tubes or hot wire anemometers may have to be inserted through the walls. In other cases, the injection of dyes or fine streams of air bubbles may be used to show flow patterns.

Flow-measuring devices such as Venturi meters, orifice meters, or weir boxes are required for determining the rate of flow. Much depends on the accuracy of these flow measuring devices and carefully calibrated, properly placed equipment is essential. Quality metering devices are available and the selections can be based on accuracy desired, cost, convenience of use, and expected service life.

Properly fitted, tightly connected pipes, fittings, and transitions should be used between the model and the flow-measuring device to insure that all measured flow reaches the model test section. This requirement applies equally to water and air models. Leaks in air models are often difficult to detect because they leave no visible trace. In water models, outside leakage and drips are easily detected, but leakage of air into zones of subatmospheric pressure is less obvious.

In assembling the model components accurate alinement must be maintained. Adequate supports, provided beneath the model, should be placed to provide minimum interference when alterations to the model are made. Usually it is convenient to place the model at waist height for ease of construction, operation, and observation.

MODEL OPERATION

The testing program must be carefully planned to make full use of the model. Preliminary runs are necessary to reveal defects, leaks, or other inadequacies. First tests also indicate whether or not minor changes are needed to make the model and the instrumentation better fitted for obtaining the desired data. This phase should not be hurried because it is economically important to be certain the model really represents prototype conditions and that the instrumentation is satisfactory. It is during this adjustment phase that the experimenter becomes acquainted with the peculiarities of the model and adept in its operation. Testing should consist of examining systematically each proposed, feasible design for possibilities for improvement in flow conditions, cost reduction, and ease of maintenance. The experimenter must exercise patience, imagination, and ingenuity and be capable of interpreting the model results correctly. The data should be analyzed concurrently with the testing to prevent accumulation of unnecessary data, and to permit intelligent decisions concerning succeeding steps in the testing program. A relatively complete and detailed diary of the study is important. This diary should include dates, general notes on performance, changes made, the model data, and photographic records. Often an appreciable period of time will elapse between the time of a model study and the time when a report is prepared. Without a record such as the diary provides, many factors could be forgotten or overlooked.

Measurements of pressures, discharges, velocities, etc., must be made with precision. Much depends on the values recorded on the data sheets, and upon pertinent remarks such as pressures being particularly unsteady at certain locations. The type of pressure and velocity records made depends upon the use intended for the data. In many cases, small pressure fluctuations are

not significant and the average pressures are the ones desired. Damping devices may be used in these pressure lines to make the reading simpler. Other cases will occur where transient pressures are desired. These measurements must be made with special equipment such as pressure cells and electronic recorders that are capable of following the pressure swings. No appreciable damping should occur during these measurements. In all pressure readings, datum points or gage zeroes must be recorded so that actual pressure values at the piezometers will be known. The testing must be extended over the entire range of equivalent discharges and operating conditions that will occur in the prototype. Frequently it is desirable to extend the range of the tests to heads much higher than the equivalent heads anticipated for the prototype to fully exploit the model.

In addition to routine testing of immediate interest, the experimenter should be alert to obtain general information that will be of value at a later date. Results of experiments on many models are required for assembling general design information. Time and money may be wasted if all the useful information is not obtained from each model under test.

Studies made using air as a flowing fluid usually require that the flow velocities be maintained below approximately 300 fps. If higher velocities are used, compressibility would have to be considered. The density of the air will vary from day to day with changes in temperature and/or barometric pressure. Thus, if pressures are read on water manometers in feet, and are converted to feet of flowing fluid (air), the density of air relative to that of water must be known. The air temperature and barometric pressure must therefore be recorded during air studies.

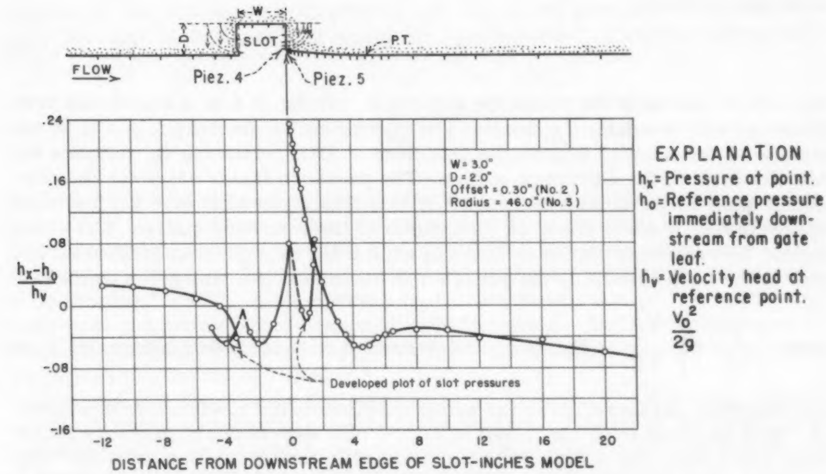
INTERPRETATION OF RESULTS

If care is taken to operate the model at Reynolds numbers of about 1×10^6 or more, the data may be analyzed using the same relations as for models based on the Froude law. Thus, direct geometric scaling of linear values is possible. If the model were run with scaled heads and discharges, the model pressures would be multiplied by the scale ratio to obtain prototype pressures. Time intervals and flow velocities are obtained by multiplying model values by the square root of the scale ratio. The discharge is obtained by multiplying the model flow by the five-halves power of the scale ratio.

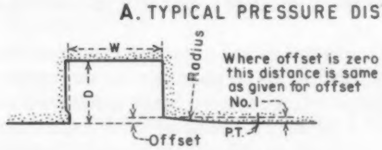
If the model is tested at heads and discharges that differ from scaled prototype values, the data may be placed in a dimensionless form and then applied to prototype conditions. An example is shown in Fig. 10. A typical dimensionless pressure factor relation is

$$\left(h_x - h_0\right) \Big/ \left(V_0^2/2\,g\right)$$

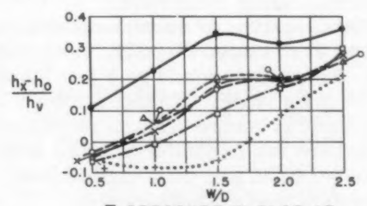
where h_x is the head at the piezometer in question, h_0 is the head at a reference piezometer, and V_0 is the Q/A velocity at the reference station. The reference station is usually taken at a convenient point just upstream from the test section itself. Other pressure factor relations may be used as conditions dictate, provided they involve a pressure relationship from a base point and a parameter, such as the velocity head, that describes the flow.



A. TYPICAL PRESSURE DISTRIBUTION CURVE



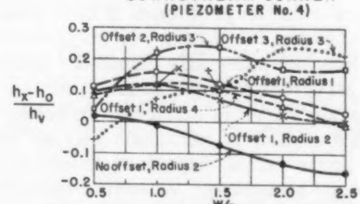
B. DIAGRAM OF SLOTS



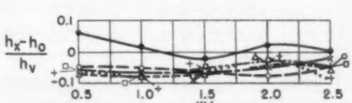
E. PRESSURE IN SLOT AT DOWNSTREAM CORNER (PIEZOMETER No. 4)

W/D	0.5	1.0	1.5	2.0	2.5
OFFSET NO. 1	0.15W	0.075W	0.05W	0.037W	0.03W
OFFSET NO. 2	0.30W	0.15W	0.10 W	0.075W	0.06W
OFFSET NO. 3	0.45W	0.225	0.15W	0.112W	0.09W
RADIUS NO. 1	12.6W	6.30W	4.20W	3.15W	2.52W
RADIUS NO. 2	42.5W	21.25W	14.167W	10.625W	8.50W
RADIUS NO. 3	46.0W	23.0W	15.33W	11.50W	9.20W
RADIUS NO. 4	97.8W	48.90W	32.60W	24.45W	19.56W

C. OFFSETS AND RADII RELATIVE TO SLOT WIDTH



F. PRESSURE BELOW DOWNSTREAM CORNER OF SLOT (PIEZOMETER No. 5)



D. LOWEST PRESSURE IN SLOT

G. LOWEST PRESSURE ON CURVED CONVERGING WALL

FIG. 10.—PRESSURE CHARACTERISTICS OF A GATE SLOT

Data in the foregoing dimensionless form is applied as follows: Determine from the relation

$$Q = C_d\ A\ \sqrt{2\ g\ H} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

the rate of flow Q in the prototype structure. In Eq. 2 A is a significant area in the gate or conduit, C_d denotes the coefficient of discharge, and H is the total head differential across the structure. After obtaining Q, compute the velocity head at the reference station. The pressure factor value for the particular piezometer is then multiplied by this velocity head to give the pressure change in feet of water from the piezometer to the reference station. The pressure at the reference station is found by adding any back pressure on the structure to the head caused by the presence of the structure, and given by the relation $\left(\frac{1}{C_d^2} - 1\right) \frac{V_0^2}{2\ g}$. The computed pressure change, added algebraically to the reference pressure, gives the prototype piezometric pressure to be expected. Data obtained from air model tests are best expressed in pressure factor form, using care that all values are expressed in feet of the flowing fluid. The pressure factors are dimensionless and are applicable directly to prototype conditions using other fluids.

Often negative, or subatmospheric, pressure heads are obtained from models which, when scaled directly, yield values too low to be possible in the prototype. An example would be a 3-ft subatmospheric pressure head measured on a 1:20 scale model. Direct scaling yields a prototype head of 60 ft below atmospheric pressure. This is unrealistic because no pressures lower than absolute zero can be obtained. At sea level, this is about -34 ft of water. Actually, water begins to boil when the pressure is reduced to vapor pressure, about 0.7 ft absolute, and cavitation occurs. Thus, any scaled-up model pressures that are subatmospheric to the extent of about 30 ft to 33 ft indicate that cavitation will occur in the prototype. If cavitation pressures are indicated over appreciable areas of the structure, large-scale separation is likely and the flow pattern may be materially changed. Specialized testing is necessary to evaluate such structures. In the usual case, the mere suggestion of pressures near the cavitation range is sufficient to justify redesign so that noise, vibration, and damage caused by cavitation will not occur.

In some cases, negative pressures of limited extent may be acceptable in prototype structures. When flow velocities and pressure fluctuations are moderate or small, indicated pressures 15 ft below atmospheric are not considered objectionable. It is usually dangerous to allow pressures lower than this because unexpected surface roughnesses, vorticity, and flow turbulences may momentarily lower local pressures to the cavitation range.

Studies of cavitation potential on particular structures are frequently made and often require special test equipment and instrumentation because of the complexity of the problems. Certain parameters are commonly used to express cavitation potential on a structure. One is the cavitation index

$$K = \frac{h_o - h_{vp}}{\frac{V_0^2}{2\ g}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

in which h_0 is the pressure at a reference point, h_{vp} is the vapor pressure relative to the atmosphere (negative), and V_0 is the velocity at a reference point. Another index especially useful for systems such as valves in pipelines is

$$K = \frac{h_2 - h_{vp}}{H_T - h_2} \qquad (4)$$

in which h_2 is the pressure in the pipeline well downstream (for instance a distance of 12 D), h_{vp} is the vapor pressure, and H_T is the total head upstream. This parameter has the advantage of describing pressure conditions which must exist with given flow velocities and valve characteristics if cavitation is to be avoided.[8] With either parameter, for the same value of K in model and prototype, the pattern of cavitation will be the same. But since frequency of vibration and rate of pitting vary with velocity, care must be used in predicting prototype structural behavior.[9]

The head loss through the system is obtained by taking pressure and velocity measurements at appropriate points in the system. Similarly, the coefficient of discharge for the structure may be obtained with appropriate measurements of pressure, discharge, and valve or gate opening. This coefficient is commonly expressed as

$$C_d = \frac{Q}{A\sqrt{2gH}} \qquad (5)$$

in which Q is discharge, H is head drop across the structure, and A is a defining area similar to that of the conduit, the control body, or the opening beneath the gate leaf. Force measurements, such as downpull on a gate leaf, torque on turning vanes, or thrust on walls, may be obtained by applying measured pressure heads on areas that will be affected, or by making direct force measurements. If the latter method is used, care must be taken to eliminate unwanted model friction and extraneous vibration.

The usefulness of air models in studying closed conduit systems has been discussed. Such models discharge into the atmosphere and therefore operate "submerged," which is analogous to a water model discharging under water into a large pool. Air-model data must therefore be interpreted as submerged flow. This is no handicap in closed conduit systems, but must be recognized and accounted for if model studies are made on prototype structure such as a gate that discharges water freely to the atmosphere.[10]

Hydraulic models sometimes combine both open and closed conduit flow conditions. An outlet works illustrates a case where studies are made of the control valve or gate and of the stilling basin downstream. Both the open and closed conduit portions of the model are analyzed by the Froude relationships, if the operating Reynolds numbers are in the appropriate range.

It is difficult to build a model with equivalent dynamic response to that of the prototype. This does not preclude making studies of the stability of the system or reasonable predictions as to whether or not resonance and vibration

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8 "Cavitation Characteristics of Gate Valves and Globe Valves Used as Flow Regulations under Heads up to about 125 Feet," by J. W. Ball, Transactions, ASME, August, 1957.

9 "Cavitation Scale Effect," by R. T. Knapp, Proceedings, Seventh Genl. Meeting, I. A.H.R., Lisbon, 1947, A6.

10 "Air Model Studies of Hydraulic Downpull on Large Gates," by W. P. Simmons, Jr., Proceedings, ASCE, Vol. 85, No. HY 1, January, 1959.

will occur. Simultaneous recordings of pressure cell measurements at appropriate points within a system can show if pulsations and/or surge will be present. These data may be applied mathematically to the prototype structure for analysis of possible resonance. In such tests, particular care must be taken to supply water to the model without pump or other mechanically induced pressure fluctuations. Pressure cell records will be required on the incoming pipeline to show whether or not such pressure fluctuations exist. Special and detailed data analysis are required if such fluctuations are entering the system to complicate the pressure conditions.

Graphs, charts, and other plots are usually needed to analyze and illustrate the data obtained. Final results find more usefulness when expressed in dimensionless parameters that may be applied to other geometrically similar structures. Plots showing specific pressure conditions and other characteristics applicable to the structure under study are also made.

Reports constitute the chief instrument through which the benefits of the model study are transmitted to designing engineers, and to the profession in general. They must be complete, accurate, timely, and to the point. However, brevity is no virtue if attained at the expense of completeness, clearness, or accuracy.

CONCLUSIONS

1. An understanding of Reynolds similitude factors, and proper application of them in designing, operating, and interpreting closed conduit hydraulic models will lead to reliable and accurate predictions of prototype performance.
2. Models used in the usual studies of hydraulic control elements and conduit systems need not be excessively large, or operated with extreme flow velocities to yield reasonable model-prototype similitude. As a rule of thumb, a model operating at or near a Reynolds number of about 1×10^6 is adequate.
3. Surfaces of model flow passages must usually be smoother than the prototype surfaces for friction to be similar. In short segments or portions of conduit systems, full similitude in this respect is not important, and good results will be obtained if the velocity distribution entering the section is turbulent and fully developed. In models with long water passages, more effort may be required to obtain friction similitude, if required, because friction would be of relatively great importance.
4. Particular care must be taken to produce accurately shaped flow surfaces, and to maintain precise alinement of the model components.
5. The selection, location, construction and maintenance of measuring devices, such as piezometers and recording instruments, must be done with care and a knowledge of the type of data to be obtained.
6. Tests should be continued to the point of obtaining all feasible information from each model so that the time and expense of the model construction will be exploited fully.
7. Detailed diaries or logs should be kept of the studies, and comprehensive reports prepared to make the material available to the engineering profession.

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

(Over)

Note.—This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 6, June, 1960.

HYDRAULIC MODELS OF THE ST. LAWRENCE POWER PROJECT[a]

Closure by J. B. Bryce

J. B. BRYCE.[1]—The author wishes to express his appreciation of the discussions prepared by Mr. G. T. Berry and Messrs. G. B. Fenwick and F. R. Brown, both of which contained much additional useful information. As mentioned previously, the comprehensive nature of the paper necessitated a general coverage of the subject and space did not permit treatment in detail of the various problems encountered. However, in both discussions, interest was shown in the design of energy dissipation works and more information was requested in connection with the investigation of hydraulic loading of gates.

With regard to the energy dissipation at the Iroquois Control Dam, Messrs. Fenwick and Brown note that only 10 of the 32 sluices were thus protected. As they surmise, very substantial economies were effected by this partial treatment. A careful analysis supplemented by model tests indicated that only during construction would an energy dissipator be required, and in this high head period the flow could be confined to 10 sluices. This was later corroborated in actual operation. These discussers also note that the pattern of gate openings can have a considerable effect on energy dissipation and in the magnitude of the currents downstream. This was forcibly illustrated at Iroquois during construction and even after the power pool was raised. Initially during these periods, fairly severe currents were observed below the dam which proved troublesome to navigation. A very intensive series of model tests was run in the Iroquois Dam model, and a pattern of gate openings was developed to maximize energy dissipation and minimise downstream currents. When these patterns were put in operation at the dam a remarkable improvement occurred and conditions acceptable to navigation were produced.

Messrs. Fenwick and Brown raised the question as to the necessity of a 40-ft wide end sill for the Long Sault Dam dissipator and whether a longer apron and shorter sill might improve conditions. As mentioned in the paper, over thirty different designs were tested which included several combinations of the above variables, and it was considered that the design selected was the best in view of all the circumstances. Some of the factors which influenced the selection were: 1) the depth of the bucket was limited from geological considerations; 2) it was desired to have the apron as short as possible, consistent with satisfactory performance; and 3) as the issuing velocities were still rather high, the end sill was made wider than hydraulic conditions alone would require in an effort to ensure that, if undercutting did take place, sufficient sill would remain to function properly.

Both Mr. Berry and Messrs. Fenwick and Brown requested further details regarding the investigation of hydraulic loading on gates. Although space will

[a] May, 1959, by J. B. Bryce.

[1] Hydr. Engr., Hydro-Elec. Power Comm. of Ontario, Toronto, Canada.

not permit a complete discussion of this subject, the writer feels that certain information additional to that given in the paper might prove useful. Accordingly, and with reference to the original paper, the following remarks are appended.

Port Gates at Long Sault Dam.—The results of the tests on these gates are given in Fig. 16 of the paper. The gates were 22 ft, 6 in. high, 17 ft wide, and 2 ft 4 in. deep, and had the skinplate on the downstream side. The tunnel ports were 18 ft high at the upstream and 15 ft high at the downstream end and were 16 ft wide. Each gate was to be capable of shutting off a maximum flow through the port of 14,000 cfs travelling at an average velocity of 40 fps and was to be fully submerged in operation. All tests on the gates were carried out at a head of 77 ft on the sill and with free discharge conditions at the outlet.

Two points not mentioned in the paper might prove of interest. The first is that when tests were carried out on the original gate design, the flat bottom caused the spring point of the flow to move from the downstream edge towards the upstream side under certain conditions. When this took place an oscillating vibration of impressive proportions occurred. This behaviour is in agreement with that cited by Messrs. Fenwick and Brown in their discussion. The second point is the effect on the downpull of a recess made in the upstream face of the dam above each port and immediately back of each gate when in the wide open position. The recess in each case was 16 ft wide, 12 ft high and 1 ft deep, and it was found to reduce the downpull by 12 tons. It is believed that the reduction in downpull by the recess is due to balancing the pressure across the top seal. However, if the recess was extended beyond the top of the gate in the fully open position, there was a tendency for the gate to move outwards from the dam. The significance of the clearance on the downstream side of the gate was also mentioned by Messrs. Fenwick and Brown.

Following the tests reported in the paper, an additional investigation was made to determine the effect of several variables. In brief it was found that, without air venting, downpull varies directly with head, and that air venting is increasingly effective in reducing downpull as the head increases. It was also found that approach conditions have a considerable effect on the vertical forces in the gate. For example, lowering the bed upstream some 30 ft reduced the downpull by over 20%.

Powerhouse Headgates.—The headgates tested were those for the Canadian powerhouse which are of somewhat different design than those for the United States powerhouse. Fig. 18 shows the gate dimensions and the details of one of the three water passages leading to the scroll case. The gate skinplate is on the downstream side and the gates operate in a fully submerged condition. It was determined that the critical conditions for hydraulic loading on the gate would occur under runaway conditions with two of the inlets completely closed before the third gate started downward. The tests were performed with constant headwater and tailwater levels. The wicket gates were set to pass 14,000 cfs at the beginning of the test (runaway discharge) and locked in position.

Two of the passages were closed leaving the gate under test to close the remaining opening which was found to be carrying about 13,000 cfs at 20 fps when the test commenced. In Fig. 18 the results show that the initial downpull of 17 tons increased to a maximum of 119 tons at a gate opening of 8 ft. This may be compared to a gate weight of 37 tons. It was considered that the gate bottom was sloped as much as structural design would permit and no reduction in load was found possible.

Massena Intake Gates.—At the Massena Intake, two gates were tested, the emergency gate which had a skinplate on the downstream side, and the service gate which had the skinplate on the upstream side. Fig. 19 shows the arrangement and dimensions of the gates and the details of the water passages. The tests were carried out at a constant headwater level and two selected tailwater levels. The maximum test flow was 21,000 cfs producing velocities of over 40 fps through the port.

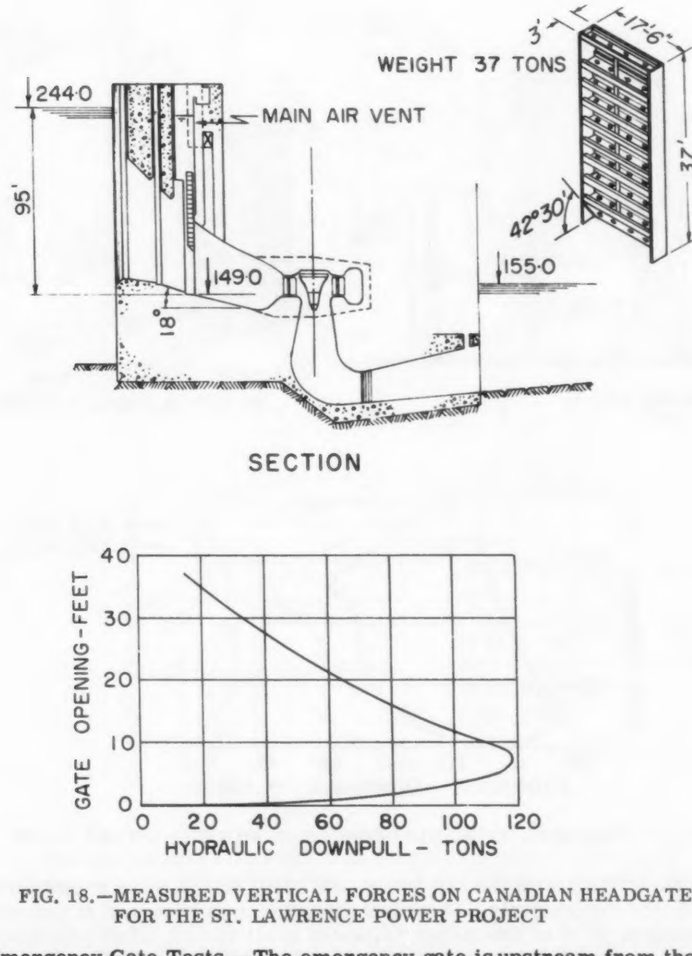

FIG. 18.—MEASURED VERTICAL FORCES ON CANADIAN HEADGATES FOR THE ST. LAWRENCE POWER PROJECT

Emergency Gate Tests.—The emergency gate is upstream from the service gate and is completely submerged in operation. As shown in Fig. 19, maximum downpull forces of 68 tons and 76 tons were measured for the two heads tested. It may be noted that in the fully open position a downpull of about 50 tons was measured, or about 140% of the gate weight. Comparing these downpull

forces with those measured on the Long Sault Dam port gates it would appear that they are somewhat greater than would be expected. It is believed that the two horizontal sections in the bottom, which accommodated the wheel, axles, increased the downpull force. Also, this gate is 50% higher than the Long Sault gate and the drag forces are probably greater.

Service Gate Tests.—While the service gate is similar in size to the emergency gate, and the test conditions were identical, the results of the tests were greatly different as shown in Fig. 19. Downpull forces amounted only to 10 and

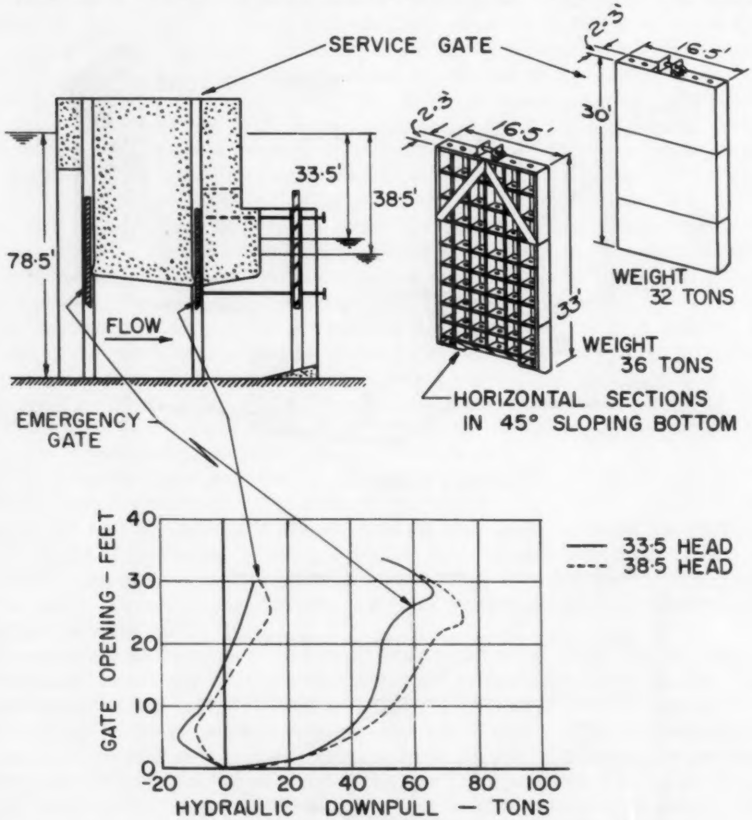

FIG. 19.—VERTICAL HYDRAULIC FORCES ON MASSENA INTAKE GATES

15 tons respectively with the two heads, but uplift forces were encountered of 16 and 10 tons respectively. A violent vibration was observed at and near a gate opening of 25 ft at the higher tailwater level tested, when variations in force of plus or minus 35 tons were observed. This tendency was reduced but not eliminated at the lower tailwater level. It is believed that the uplift was due to the considerable back-pressure on the gate from the tailwater, and that the vibration was due to unstable conditions at certain gate openings when a rather violent pumping action took place in the tailwater.

Iroquois Sluice Gates, Long Sault Crest and Diversion Gates.—All these gates are similar in that they are 52 ft wide, 3 ft in thickness and have bottoms which sloped upward from the skinplate in the direction of flow at an angle of approximately 45°. All the gates had upstream skinplates and were unsubmerged in operation. The Iroquois gates were sheathed also on the downstream side to permit heating for winter operation. In Fig. 20 the results of all the tests and the gate dimensions and sketches of the gate settings are given. It should be noted that the head referred to is that on the sill for the Long Sault Crest

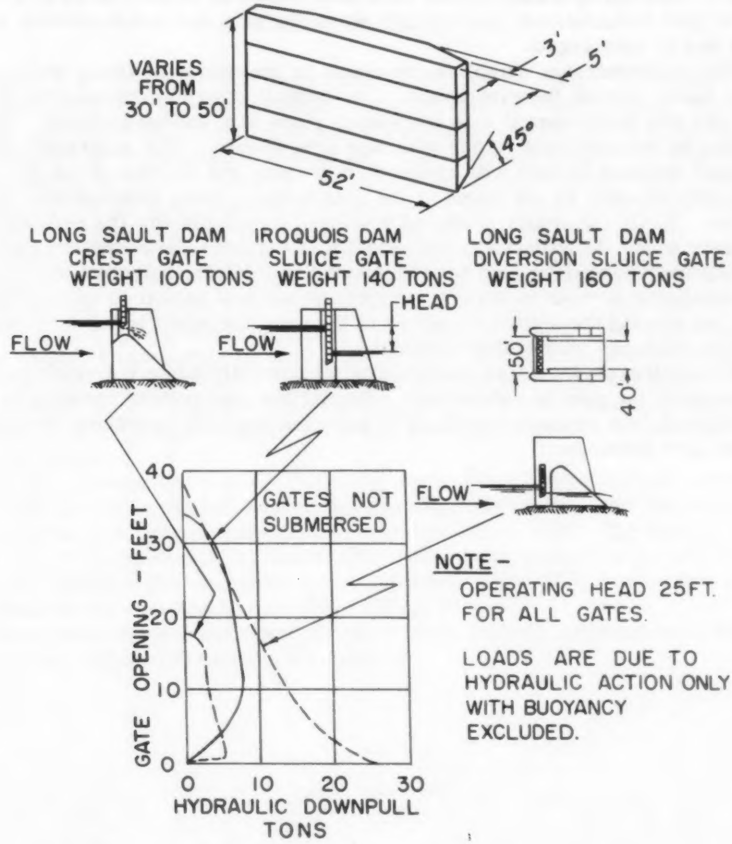

FIG. 20.—MEASURED VERTICAL FORCES ON CREST AND SLUICE GATES FOR LONG SAULT DAM AND IROQUOIS DAM.

gate, and is the differential head in the other two cases. It was concluded from the results that in the case of the Long Sault Crest gates and the Iroquois gates, the downpull was caused by drag forces only, as atmospheric pressure occurred underneath both gates at the point of maximum downpull. The increased downpull at the Long Sault Diversion gates was believed due to the restricted setting in which a great mass of water was thrown into the exposed downstream

side of the gate, thereby increasing the load. This was confirmed by testing this gate in the Iroquois setting where the downpull was greatly reduced.

To obtain some information on the effects of bottom design on gates of this type the Iroquois gate was tested with a flat bottom. In the tests, at the point where submergence of the gate bottom began, a violent vibration occurred. The maximum downpull was also increased from 8 tons to 35 tons.

While the tests previously described by no means cover the subject of vertical forces on gates, it would appear that certain general conclusions may be drawn. Vertical hydraulic forces have been shown to be very large at a number of gate installations, particularly when the gate has a downstream skinplate and is submerged.

With a downstream skinplate, downpull is the dominant force which may many times exceed the gate weight. Substantial downpull forces may occur with the gate fully open if it is suspended close to a submerged port. Uplift appears to be relatively minor with this type of gate. The magnitude of the downpull appears to vary with the head on the gate and the size of the gate and is greatly affected by the shape of the gate bottom, being greatest with a flat bottom. While the major cause of downpull is undoubtedly the reduction in pressure under the gate due to the velocity, it has been shown that drag forces and approach conditions also have an appreciable effect. Effective air venting and equalizing pressures across a projecting top seal appear to reduce downpull, but sloping the bottom members of the gate downward in the direction of the flow produces the greatest reduction.

With upstream skinplates, downpull is not normally large, but uplift may be important if the gate is submerged. Sloping the gate bottom upwards in the direction of flow appears beneficial in reducing unstable operating conditions at part gate openings.

GROUND-WATER PROBLEMS IN NEW YORK AND NEW ENGLAND[a]

Closure by Joseph E. Upson

JOSEPH E. UPSON.[1]—The discussions of the writer's paper by Messrs. C. Biemond and Robert O. Thomas reveal two quite different categories of ground-water problems. Mr. Thomas' comments elaborate on the need for adequate investigation of the geologic, hydrologic, and economic features pertinent to the development of ground-water resources. Solutions to these general problems are certainly fundamental in the early stages of planning a water-supply installation, whether it be for irrigation, industry, or domestic supply.

In contrast, Mr. Biemond points out a much more specific type of problem which lies in the narrower realm of the minute relationships between ground water, with its dissolved, suspended, and bacterial loads, and the small-scale environment of its enclosing deposits. The effects of these relationships sometimes appear in operation of an installation and can be almost as critical as the amount of available water. Both categories are applicable and pertinent to development of the ground-water resources of New York and New England. Few workers with, and students of, water are knowledgeable in the fields of both categories.

For the readers who may not fully appreciate Mr. Biemond's final reference to advantages of artificial replenishment, the Amsterdam, The Netherlands, water works has been pumping water from the Rhine within The Netherlands and, with minor pretreatment, infiltrating it into a segment of the ground-water reservoir beneath The Netherlands coastal dunes since 1957. Apparently there has thus far been no clogging or other difficulties.

The writer extends thanks and appreciation to Messrs. Biemond and Thomas for interesting and informative discussions.

---

[a] June, 1959, by Joseph E. Upson.

[1] Research Geologist, Ground Water Branch, Water Resources Div., U. S. Geol. Survey, Mineola, N. Y.

AN ENGINEERING APPRAISAL OF HYDROLOGIC DATA[a]

Task Group on Hydrologic Data of the Committee on Hydrology.

DONALD W. VAN TUYL,[1] F. ASCE.—The thorough explanation by Mr. Kazmann of "estimated" and "safe" yields of ground water aquifers should greatly assist those considering the types of hydrologic data reviewed in the report of the Task Group on Hydrologic Data. Classifying "estimated yield" as interpretive data was intended to illustrate the distinction between quantities determined by compiling other values and quantities determined by more or less routine measurement or analysis. Primary emphasis was given throughout the report to basic and analyzed data with only bare mention of the interpretive classification.

The two terms which Mr. Kazmann discusses are used in ground water computations without exact definition, as he clearly indicates. Both may be found to have specific values during engineering analyses, however, which may also involve economic factors.

The term "interpretive data" in the Task Group report should be viewed as an intermediate product of raw materials (data) but not an end in itself—a value or item of knowledge to be used with other items in obtaining a further answer. An item of data that results from interpretation and extrapolation of basic data and is employed in subsequent hydrologic applications, therefore, may be classed as interpretive under this concept.

For example, a computed flood stage for a particular frequency is the result of numerous compilations of basic and analyzed data, but it is meaningful only when applied as an item of data to an engineering problem. A member of the Task Group suggests that storage-yield curves for a surface water development are also interpretive data.

One reference in the Task Group report to another publication, unfortunately, was out of date when the report reached printed form. Reference No. 1 should read:

1. Langbein, W. B. and Hoyt, W. G., 1959, "Water Facts for the Nation's Future," The Ronald Press Co., New York; sponsored by the Conservation Foundation.

[a] July, 1959, by Donald W. Van Tuyl.

[1] Chmn., Task Group on Hydrologic Data, Committee on Hydrology, and Water Resources Asst., Chamber of Commerce of U. S., Washington, D. C.

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HYDRAULIC CHARACTERISTICS OF GATE SLOTS[a]

Discussion by C. T. Advani

C. T. ADVANI.[1]—The writer found Mr. Ball's paper most interesting, especially concerning the large number of model test results that he has been able to furnish on various different type of slots.

A certain number of the results with the different type of slots obtained by Mr. Ball agree with those found during the detailed program of investigations carried out during the last 3 yr in the laboratories of the Societe Grenobloise d'Etudes et d'Applications Hydrauliques, Grenoble. The investigations were especially concerned with the problem of partial openings of vertical lift gates under very high heads and the resulting cavitation conditions in the gate slots.

The writer would, however, like to stress that it is absolutely essential that, in order to interpret the test results obtained correctly, the similitude between the prototype and the model must be established perfectly.

If this is not done, in a number of cases the interpretations may prove to be rather faulty. Very great care on this account has to be taken in designing the test installation, so that the Froude similitude and the Reynolds number are sufficiently high. The test installation used in the laboratory had the clear gate dimensions: 600 mm high and 260 mm wide. The gate model was hydraulically operated by servomotor control. The cavitation circuit was designed to pass 1200 liters of water per sec. With this type of circuit having satisfactory pressure control, it was possible to obtain reasonably accurately the approach of cavitation breakdown. Also, in order that the results obtained are not faulty, it is essential that the circuit be clear of parasite cavitation in bends, the pump, etc., so that the observed changes in the regime are due to the gate operation and hydraulic conditions only. Further, if water with some air-content is to be used, the circuit should be provided with air reabsorber, so that bubbles do not collect in dead pockets.

The writer completely agrees with Mr. Ball that the slot problem is essentially a detailed study of pressure intensity and distribution. The heads, the dimensions as well as the downstream hydraulic conditions will determine whether there is cavitation or not. The intensity of cavitation will at the same time depend on the gate opening. For different hydraulic and slot conditions, the critical openings vary. For a number of existing installations, the gates have not shown signs of cavitation, because they have never functioned under the critical openings for the hydraulic conditions imposed.

The writer feels that the correct mode of attacking the problem was not to find a typical design which would be fool-proof under all different operating and hydraulic conditions but to study each particular case separately.

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[a] October, 1959, by J. W. Ball.

[1] Engr., Sogreah, Grenoble, France.

The series of tests that have been carried out by the writer on different types of slots and different hydraulic conditions imposed have proved that for effective heads of over 40 m with downstream undrowned (free discharge) and with typical classical vertical lift gates, freedom from cavitation cannot be had under all possible openings. This condition is also applicable for arrangements having upstream contraction and air duct tubes feeding along the height of the slots. Actually, the best arrangement is with upstream sealing and normal slots without offset. With this disposition, it would be possible to operate the gates without large cavitation risks for higher heads, but only for intermediate (between 1/3 and 2/3) and full gate openings.

If however the downstream jet is drowned (backpressure), the results are most unfavorable whatever the position of the sealing, the type of the slot, or the gate opening.

If the vertical lift gates are required to function partially open under very high heads, then a solution other than the classical gate has to be envisaged. During our investigations, it was specified that a solution should be found which should not only be suitable from the hydraulic point of view, but had to be feasible mechanically and from the economic stand point, the cost had not to exceed those of the classical vertical lift gate. The solution of baffles or extensions to bottom corner of gate leaves (in any case not a very healthy solution) or very large offsets in the slots had to be rejected. After a considerable amount of study and tests, a gate design was evolved which satisfied the different conditions imposed.

A brief mention of this solution was made by Messrs. Zaky and Sauvage de Saint Marc at the last Congress of I.A.H.R.

It is impossible to give a complete idea of this solution in this short note of comments. Briefly speaking, the gate leaf knife edge is designed to be upstream of the gate slot, it being supported on a sort of parabolic bridge structure. The slots from the hydraulic point of view are nearly in uniform flow without large variations in pressure along the height of the slot. The results that were obtained during the tests were most encouraging.

In case of partial gate openings, just downstream of the gate, one has over the resulting jet surface either air (free surface flow) or a mass of water (drowned jet) in which the pressure distribution is practically hydrostatic. In case of free surface downstream, the h downstream (back pressure) is the height of the jet itself above the gate sill. In case of drowned jet, the h downstream is the height of water in the air duct, also based on the sill level of the gate.

To obtain accurate results, very delicate pressure measurements have to be taken. An exact idea of pressure fluctuations is essential. To know the average pressures is not enough.

The cavitation coefficient can be described as:

$$\sigma = \frac{\text{h downstream} + \text{h atmospheric} - h_v}{\text{h upstream} - \text{d downstream}}$$

The numerator is the difference between the absolute pressure downstream of the gate and the vapour pressure h_v. The denominator is $\frac{v^2}{2g}$ of the jet, which is in itself the difference between the pressure just on the upstream and downstream side of the gate. The sill is taken as a datum for all heads.

The classical vertical lift gates with upstream sealing and normal slots without offset and downstream undrowned cavitate around $\sigma = 0.25$ at difficult openings (less than 0.2 and greater than 0.7) and at $\sigma = 0.1$ at favorable openings. With downstream jet drowned, the σ is rarely under 1.

The writer trusts that the comments give at least a rough idea of the way the problem of gate slots has been studied and that they will help to compare with the results obtained by Mr. Ball.

DISCHARGE FORMULA FOR STRAIGHT ALLUVIAL CHANNELS[a]

Discussions by C. Blanchet, D. R. Dawdy and R. W. Carter, M. Gamal Mostafa, and James K. Culbertson and Carl F. Nordin, Jr.

C. BLANCHET.[1]—This is an extremely interesting investigation, from the practical point of view, because it provides engineers with an easy homogeneous calculation method that covers nearly all the experimental results known today, which is not the case with a certain number of earlier investigations. Like earlier publications, this article shows that the problem is not only very complex, but also that despite the many experimental results that are known and applied, their number and organisation are still grossly insufficient.

In spite of this well known weakness, that affects us all, the authors—like many of their predecessors—have given in to the temptation to generalise the solution and to give it a vaguely theoretical appearance. They have, for instance, considered parameters for which little experimental data, or none at all, are available. The results are expressed in terms of such parameters as grain shape, grain and fluid specific gravities, viscosity, gravity and temperature, none of which have really been considered in the tests under analysis. Some doubts therefore remain as to the reliability of the results, if these are precisely the parameters to be applied.

For instance, although the introduction of the drag coefficient C_d deduced from the fall velocity of a grain in still water is probably valid for the more or less spherical grains used in the tests, it is doubtful whether it would still apply, if, say, pins were used instead. Under these conditions, the form effect of an isolated pin under free fall conditions would probably bear no relationship to that of a pin lying flat on the surface of the water.

Considering a sphere, it is also doubtful whether the sudden change in the drag coefficient occurring when

$$\frac{Wd}{V} = 2 \times 10^5 \qquad (1)$$

under free fall conditions in calm water is also valid for a bed of spheres. The difference between the state of the flows and the geometry of the system is too great. In the writer's view, the introduction of the fall velocity of an isolated grain is not representative of the form effect of the same grains when forming a bed.

A further doubtful result, which is probably due to the fact that the density of the grain has not been considered over a wide range, is that the apparent weight of the grain in water does not affect the velocity under turbulent conditions (Eq. 26). In other words, for the same rate of flow, the same slope, the

[a] November, 1959, by Hsin-Kuan Liu and Shoi-Yean Hwang.

[1] Chf. Engr., Sogréah, Grenoble, France.

same canal, and grains of the same diameter but of widely varying specific gravities, (with a high sediment load in one case and none in the other) the velocity is supposed to be the same. Is this really so? Or could it be that this result was obtained because the tests considered were not carried out over sufficiently wide specific gravity and sediment load ranges?

As the title of the article suggests, it is considered that the results of the investigation are very true for natural alluvial channels, in which such parameters as gravity, grain and liquid specific gravities, grain shape, viscosity and temperature only vary slightly. However, contrary to the apparent possibilities shown in the presentation of the results, some doubts arise as soon as one leaves the scope of the basic experiments. According to the author's conclusions, it should be possible to settle the question once and for all by carrying out tests on a very wide scale.

The writer feels, however, that such things as grain shape, specific gravity, sediment load and viscosity should be investigated very thoroughly. Tests should be carried out with a variety of shapes like pins, discs, lenticular forms, and so forth, with relative densities varying between 1 to 100, and with very high sediment loads.

In this connection, we would like to draw attention to an article by M. Rottner in "La Houille Blanche," n⁰ 3 (June, 1959). M. Rottner collected 2,500 experimental results and succeeded in classifying them in terms of the following parameters:

$$\frac{d}{D'}\sqrt{\frac{S}{\frac{\gamma_s-\gamma}{\gamma}}}, \quad \frac{V}{\sqrt{h}\quad\frac{\gamma_s-\gamma}{\gamma}}, \quad \sqrt[3]{\frac{G}{\sqrt{\frac{\gamma_s-\gamma}{\gamma}}\;\gamma_s\sqrt{h^3}}},$$

in which G is the sediment load, and the remaining symbols are as in the article under discussion.

M. Rottner's investigations not only enables the same determination to be made, but also gives the sediment load. It is thus more complete and opens up

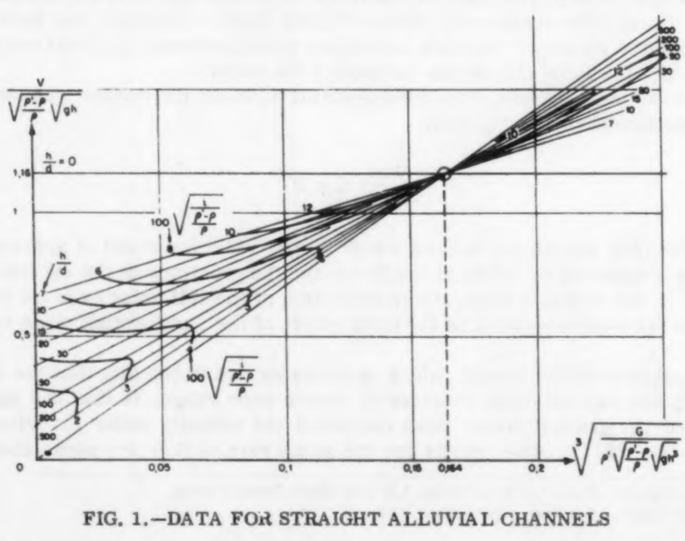

FIG. 1.—DATA FOR STRAIGHT ALLUVIAL CHANNELS

new prospects on a certain number of peculiarities and on zones so far inadequately investigated by empirical methods. The graph obtained, Fig. 1, shows certain peculiarities that will readily be noted by specialists in the subject.

D. R. DAWDY,[2] and R. W. CARTER,[3] F. ASCE.—Mr. Liu's discharge formula for alluvial channels is ingenious, but his basic assumption that various empirical flow formulae can be synthesized into one rational form does not appear to be justified. This is one universe to which order must be brought not through synthesis but through catharsis.

The author used the Blasius formula for grain sizes smaller than 0.1 mm. The Blasius formula is valid only in the range of Reynolds numbers less than 80,000, which means that for $\nu = 1.12 \times 10^{-5}$ for 65° (as used by Liu) the following limitations hold, where $R = 4\ V\ D/\nu$

	R = 4,000	R = 80,000
V, in ft per sec	D min, in ft	D max, in ft
10	.0011	.022
5	.0022	.045
2.5	.0045	.09

This range of variables is not one often encountered in nature for the flat smooth bed regime. In fact, it probably is not encountered often even in laboratory flumes. In nature the flat smooth bed regime seldom occurs at depths less than half a foot, and then only for velocities on the order of 5 fps or greater. Thus the R_{min} should be on the order of 8×10^5. Actually from field experience, the smallest R known to the writers is on the order of 18×10^5.

Perhaps the major appeal of the Blasius formula is that it is a simple power function, and easily handled mathematically. Thus, simplicity seems to be the mother of invention. Other empirical formulas[4] have been offered as well as the Blasius to describe the $f \alpha R$ relation just beyond the critical R. Blasius considered a sufficiently small range of R so that a first order approximation could be used. Since in actuality the relation is not a simple power function, the range of R can be increased only by using a more complex equation. Herman proposed,

$$f = .0054 - \frac{.396}{R^{0.3}} \quad \text{. . . . .} \quad (1)$$

This equation would be more difficult to use than the Blasius and is inadequate in that it covers only the lower limit of field data. Thus, either formula is at best only an approximation, and perhaps should not be used for determining the powers of R and S for flow in open channels.

The discharge formula proposed by Mr. Liu was used to compute the velocity for three reaches of stream channel where measurements of velocity, energy slope, and depths of flow were available. A comparison of the observed and computed velocities in the dune and smooth flat bed regimes is shown on Fig. 1(a). The measured velocity for all measurements is higher in the dune regime and lower in the flat smooth bed regime than the velocity predicted by the Liu equation. The range of scatter is on the order of 500%.

[2] Hydr. Engr., U. S. Geol. Survey, Washington, D. C.

[3] Hydr. Engr., U. S. Geol. Survey, Washington, D. C.

[4] "Essentials of Fluid Dynamics," by L. Prandtl, Hafner Pub. Co., New York, 1952.

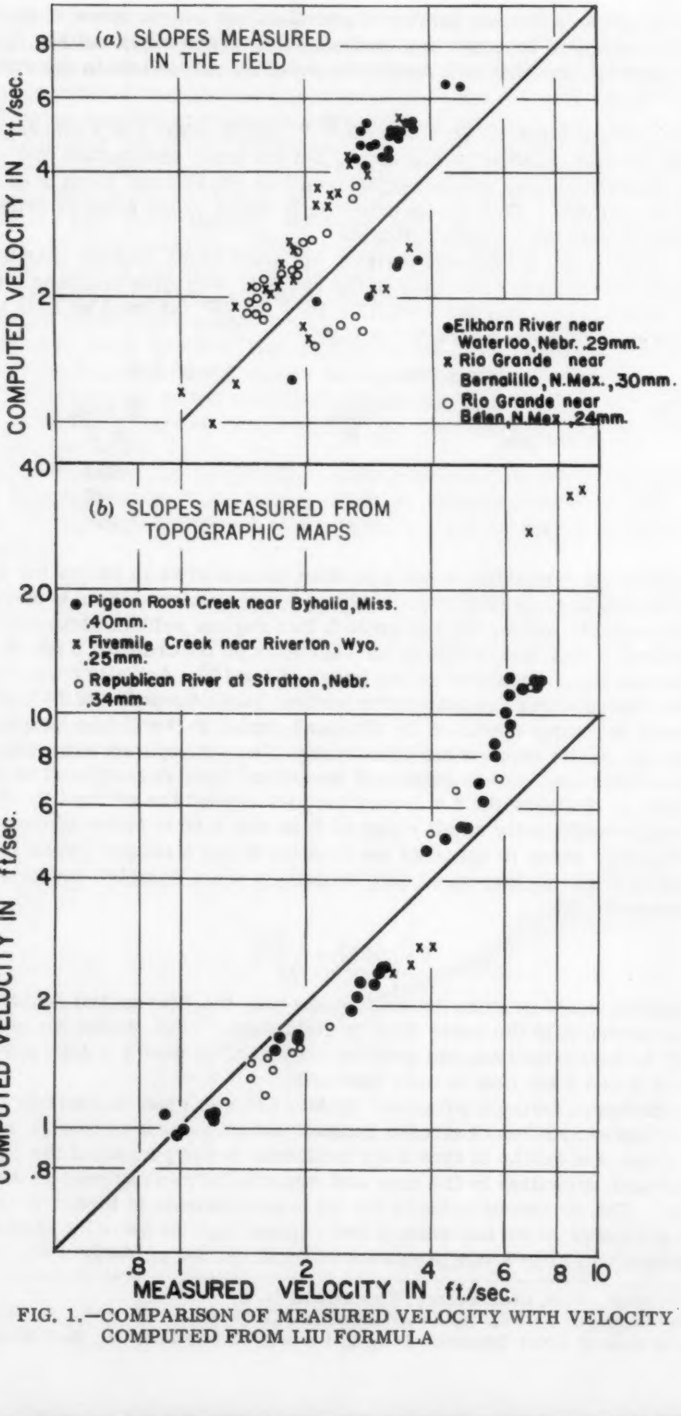

FIG. 1.—COMPARISON OF MEASURED VELOCITY WITH VELOCITY COMPUTED FROM LIU FORMULA

A similar plot for three additional reaches is shown on Fig. 1(b). For these reaches, the computed velocities for all flat smooth bed regime measurements are too high, and those for all but one dune regime measurement are too low. The slopes used in computing these velocities from the Liu equation were taken from topographic maps. The predicted velocity appears to be a better approximation of the measured velocity for the larger grain sizes.

The criterion offered for prediction of regime (Fig. 1 of paper) is based upon V_*/w and $w\,d/\nu$. This plot first was introduced in a closure to a paper by Mr. Liu[5] and therefore has not received full discussion. Wd/ν has replaced V_*d/ν in a plot Liu previously offered. Mr. Liu states, "This is permissible

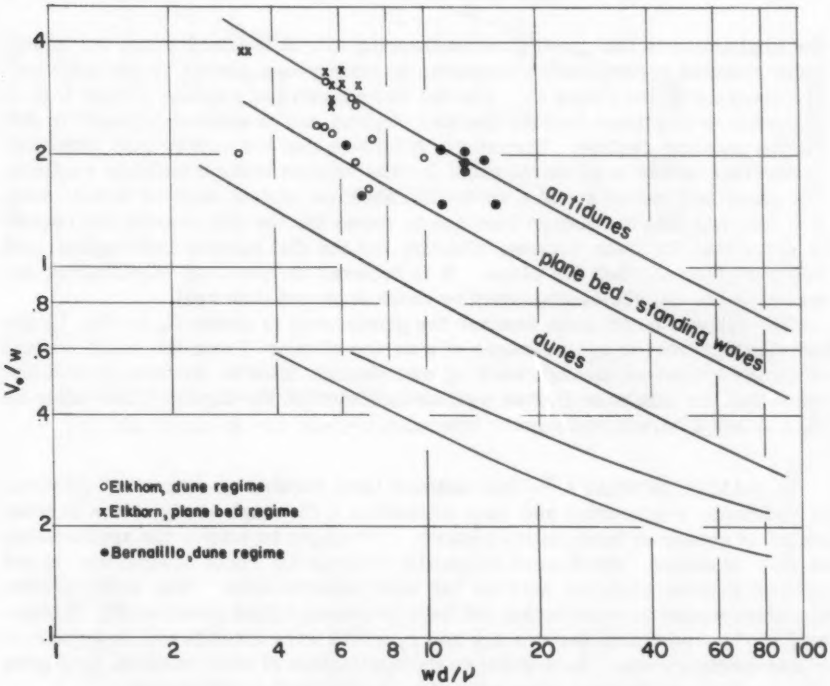

FIG. 2.—COMPARISON OF MEASUREMENTS OF KNOWN BED CONFIGURATION WITH LIU'S PREDICTED BED CONFIGURATION

because $V_* d/\nu = \frac{V*}{w} \cdot \frac{w\,d}{\nu}$." Actually this is permissible only within the meaning of dimensional analysis. Even there it is not entirely permissible, since w and d are not independent. "The mechanics upon which the criteria for various bed configurations are based . . ." does not enter into either form except a posteriori. Actually, there are six possible Reynolds numbers combining V, V_*, and w with R and d. The four involving V_* and w are identically acceptable empirically and equally unacceptable if a physical significance is implied. The term $w\,d/\nu$ is chosen for ease of use only. This field is a muddy one, and

5 "Mechanics of Sediment Ripple Formation," by H. K. Liu, Proceedings, ASCE, Vol. 83, No. HY 2, April, 1957.

mere manipulations of dimensional analysis will not make it any clearer. At any rate, the criterion offered using V_*/w and $w\, d/\nu$ does not define the bed configuration of natural channels. Data for the Rio Grande River near Bernalillo, New Mexico, and for Elkhorn River near Waterloo, Nebraska, are shown in Fig. 2 to emphasize this point.

Mr. Liu further states that the Froude number "can be considered as an index indicating the energy loss caused by the surface waves." Since

$$F = \frac{V}{\sqrt{g R_b}} = \frac{V}{V_*} \sqrt{S} = \frac{C}{\sqrt{g}} \sqrt{S} \qquad (2)$$

the implication is that for a given measuring site in the field where the energy slope remains approximately constant, the energy loss caused by surface waves is a function of the Chezy C. For the flat smooth bed regime, Chezy C is on the order of two times that for the dune regime, and is essentially equal to that for the antidune regime. Therefore, it follows that the energy loss caused by the surface waves is about the same for flat smooth bed and antidune regimes. The plane bed has no surface waves, the antidune violent surface waves. Also, it is implied that the energy loss due to waves for the flat smooth bed regime is twice that for dune regime, although for the flat smooth bed regime both water surface and bed are plane. It is believed the physical significance assigned to the Froude number may be more apparent than real.

The author should state whether the points used to define C_a in Fig. 11 are individual points, or are averages of a series of runs. From the small amount of knowledge gained through working with data for alluvial streams, it does not seem that the data ever plot as well as indicated on the figure. The scatter on Figs. 1 and 2 herein seem more typical.

M. GAMAL MOSTAFA.[6]—The authors have tackled an important problem in hydraulic engineering and have presented a thorough study of the present status of research towards its solution. Attempts to extend the applicability of flow equations, which were originally derived for rigid boundaries, to the field of alluvial streams have so far been unsuccessful. The authors have therefore turned to empiricism and have proposed a flow formula, Eq. 31, containing three sediment factors x, y and c_a which vary for different bed material and configuration. As a guide in the application of their method, they gave Fig. 1 which can be used when needed to predict bed configuration.

To estimate x, y and c_a the authors gave Figs. 9, 10 and 12, which were deduced from some collected data. The considerable jumps in the values of c_a, x and y when sand ripples appear or dunes disappear may give the impression that such change in bed configuration occur suddenly. If this was true, it is thought that there should be a critical state for such a sudden transfer. Considering the movement of a sand bed under an increasing flow traction, the change of bed shape from plane to ripple is usually noticed just after the beginning of general bed movement. The development of sand ripples is gradual and their transformation into dunes is also gradual. This was first illustrated seventy years ago, by Deacon,[7] who also noticed the gradual flattening of the

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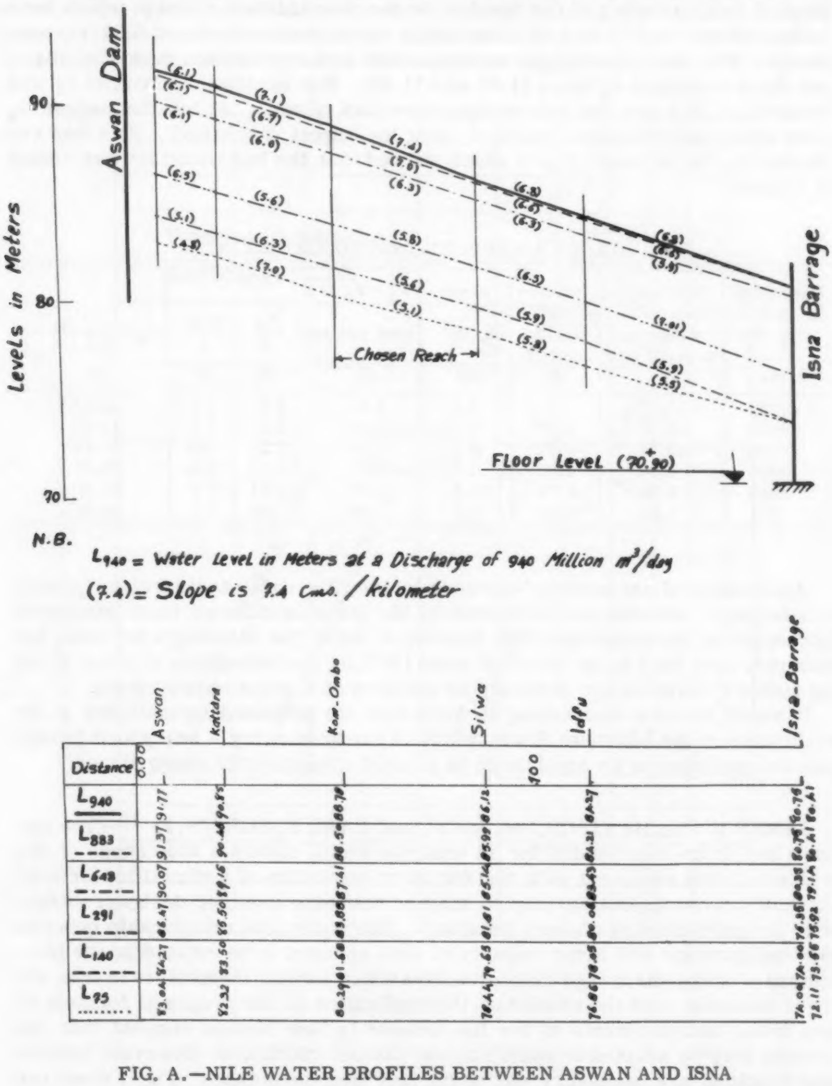
6 Dir., Hydr. Research Sta., Delta Barrage, Egypt, V.A.R.

7 "An Introduction to Fluvial Hydraulics," by S. Leliavsky, Constable and Co., Ltd., London, 1954, p. 11.

bed at relatively high flow velocities. L. G. Straub,[8] found that the maximum height of dunes was reached at critical flow conditions, corresponding to minimum specific energy, and that as the velocity of flow rose above the critical, the undulations became less and less until finally the bed became smooth.



Distance	Aswan 0.00	Koltdre	Kom-Ombo	Silwa	Idfu	Isna-Barrage
L_{940}	91.77	90.85	88.78	86.12	84.27	80.78
L_{883}	91.37	90.48	88.50	85.99	84.21	80.75 80.41
L_{648}	90.07	89.16	87.41	85.14	83.43	80.35 79.1A
L_{291}	86.47	85.50	83.80	81.81	80.04	76.35 75.92
L_{140}	84.27	83.50	81.60	79.63	78.05	74.00 73.66
L_{75}	83.04	82.32	80.29	78.44	76.86	74.04 72.11

FIG. A.—NILE WATER PROFILES BETWEEN ASWAN AND ISNA

Fig. A gives the water surface profiles of the Nile between the Aswan dam and Isna barrage at six different discharges ranging from very high to very low. During the field measurements, in each case, the discharge was kept

[8] "Discussion on Sand Movement in Fluvial Models," by L. G. Straub, Transactions, ASCE, Vol. 61, 1935, p. 867.

constant for a period long enough to maintain steady flow. A certain more or less straight reach, 30 km long, located upstream of the backwater effect of Isna barrage, was chosen for testing the authors' flow formula. Mechanical analysis of bed material in that reach gave a mean sand diameter of 0.25 mm. Table 1 is a summary of the results. In the computations x and y, which were deduced from Figs. 9 and 10 (dune bed), were taken as 0.44 and 0.31 respectively. The flood discharges corresponded to the transition region in Fig. 1 and their computed c_a were 11.61 and 11.40. For medium discharges c_a was reduced to 10.1 and the bed configuration was "dune." At low discharges c_a rose again and reached a value of 16 at the lowest discharge. This was explained by the authors' Fig. 1 which showed that the bed condition was closer to "ripple."

TABLE 1.—APPLICATION TO NILE DATA

Discharge Q, in cu m per sec (1)	Discharge per unit width q, in m per sec (2)	water depth D, in metres (3)	slope $\times 10^5$ (4)	$V_* = \sqrt{g D S}$ cms per sec (5)	$\frac{V_*}{w}$ (6)	$\frac{w\,d}{\nu}$ (7)	$C_a = \frac{q}{D^{1+x} S^y}$ (8)
10900	15.57	9.5	7.4	8.3	2.6	7.3	11.61
10200	14.57	9.3	7.0	7.95	2.5	7.3	11.40
7500	10.71	8.33	6.3	7.16	2.24	7.3	10.15
3360	4.80	4.90	5.8	5.27	1.64	7.3	10.02
1620	2.32	2.72	5.6	3.87	1.21	7.3	11.40
871	1.24	1.42	5.1	2.66	.83	7.3	16.08

Application of the authors' formula to the Nile data has therefore proved satisfactory. Discharges estimated by the formula differed from measured discharges by no more than 20%. Bearing in mind that Manning's formula, for example, may lead to an error of over 100% in the estimation of river flow, the authors' formula can certainly be considered a great improvement.

It would be very interesting to know how the proposed formula fits in its application to the Missouri River, which is known to carry a heavy load in suspension, and also in its application to alluvial streams with steep slopes.

JAMES K. CULBERTSON,[9] M. ASCE, and CARL F. NORDIN, Jr.[10]—The authors are to be commended for an analysis which covers a wide scope of the available flume and canal data, and for the presentation of a simplified formula which, if proven applicable, may be adapted with ease to either design problems or to the prediction of channel behavior. Moreover, the relationship between bed configuration and mean velocity of flow appears to be related to the phenomena of stage-discharge discontinuities which exhibit themselves in true alluvial channels, and the results of the application of the proposed formula to data from measurements of the Rio Grande in New Mexico suggest that the formula may be adapted to some natural channel conditions. However, certain modifications are necessary due to the fact that the authors' Fig. 1 does not adequately define bed configuration in the field case.

Current studies (1960) indicate that the true sand channel reaches of the Rio Grande exhibit stage-discharge discontinuities which apparently represent

[9] Hydr. Engr., U. S. Geol. Survey, Albuquerque, N. Mex.
[10] Hydr. Engr., U. S. Geol. Survey, Albuquerque, N. Mex.

changes in regimes of flow, and which may be detected by plots of hydraulic radius versus velocity. (A true alluvial or sand bed channel as used in this discussion has the same characteristics as the authors' alluvial channel, in that the bed material is granular and cohesionless and the amount of supply of sediment is equal to the amount of sediment transport.) Fig. A represents one such plot of selected measurements of the Rio Grande near Bernalillo, N. Mex. The flow may be divided into a lower and an upper regime, with a transition regime

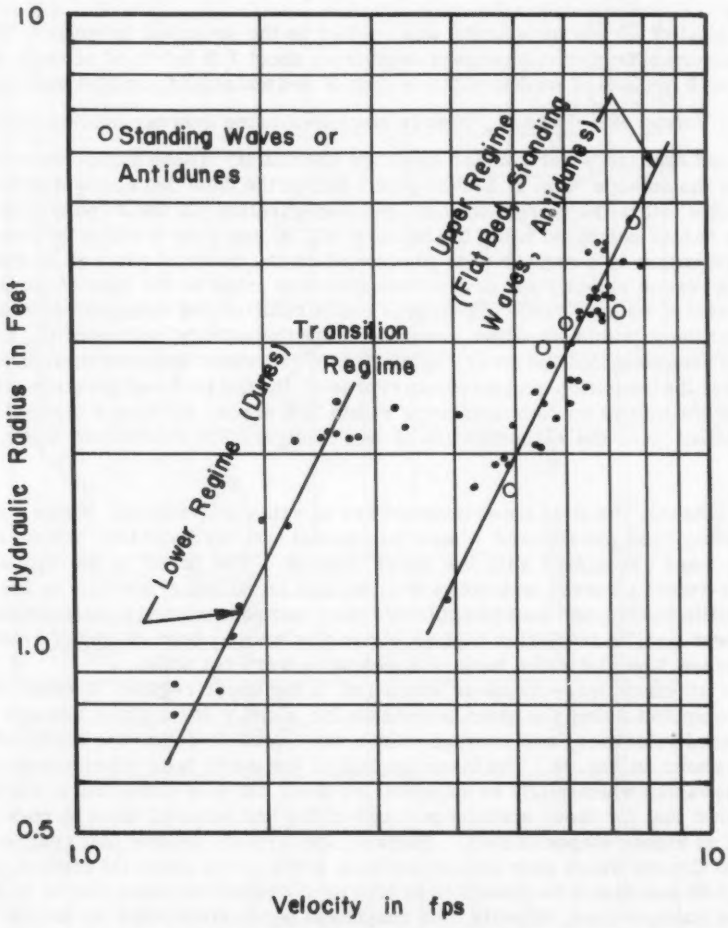


FIG. A.—RELATION OF HYDRAULIC RADIUS TO VELOCITY FOR RIO GRANDE NEAR BERNALILLO, NEW MEXICO

between. The exact nature of all changes in bed configuration which occur with changes in regimes of flow is not known, but because of the decreases resistance to flow which must accompany these changes, it is assumed that the lower regime represents flow over dunes and sand bars, and that during the transition to the upper regime, these bed features are obliterated or flattened by

the action of the flow. It was found that the velocities for points which fall along the lower regime curve may be computed using the authors' curves for the sand bar configuration, and that velocities for points in the transition stage between lower and upper regime may be computed using the proposed curves for the flat bed configuration. Velocity measurements which plot along the upper regime curve in Fig. A appear to represent flow conditions associated with a flat bed or antidunes and standing waves, and for this regime of flow it was possible to modify the C_a curve to actual field conditions, as suggested by the authors.

A total of 73 measurements was applied to the proposed formula. These measurements covered a range of depth from about 1 ft to 9 ft, of velocity from 2 fps to 8 fps, and of median particle size of bed material from 0.14 mm to 0.62 mm. Values of $\frac{w\,d}{\nu}$ and V_*/w were computed using average median particle size and average water surface slope for the reach. These values were plotted on the authors' Fig. 1. Eleven points fell in the dune bed configuration and 26 points fell in the transition (flat) bed configuration. Of the 37 points, only 6 of the values computed using the authors' C_a, x, and y were within 20% error. Classification into regime then proceeded on the basis of plots of hydraulic radius versus velocity and of field observations made at the time of the measurement of water surface appearance and of relative bed compaction. Velocities for these points were then computed using the authors' values of C_a, x, and y, and assuming that the lower regime points represent dune bed configuration and that the transition regime points represent the flat bed configuration. About 60% of the values so computed were within 20% error, showing a marked improvement over the classification of bed configuration based upon V_*/w and $\frac{w\,d}{\nu}$.

In general, the field observations were of value in indicating where standing waves were present and where the channel was hard and flat, which in all cases were associated with the upper regime. The break in the hydraulic radius-velocity curves was not so well defined for all other stations as for the Bernalillo station, and considerable difficulty was encountered in differentiating the lower and the transition regime, especially where observations of relative bed compaction and water surface appearance were not noted.

For all other measurements which fell in the upper regime, a value of C_a was computed using the given exponents for x and y for a plane bed and the measured velocities, and average values were used to define the modified C_a curve shown in Fig. B. The lower portion of the curve falls considerably below the values which might be expected for these particle sizes, and it was determined that for these stations portions of the bed scoured down to rock and gravel at higher stages of flow. Further, the writers believe that reaches of the Rio Grande which have median particle sizes of bed material greater than about 0.40 mm cannot be classified as true sand channel streams; that is to say, the bed configuration, velocity, and roughness parameters show no unique relation with change in regime of flow. This fact limits the usefulness of the curve to those reaches which meet these criteria for true sand channel streams.

The results of this study indicate that the proposed method for computing mean velocity of flow can probably be applied successfully to true sand channels such as portions of the Rio Grande, but that additional criteria beyond the relation of $\frac{w\,d}{\nu}$ to V_*/w are needed to define adequately bed configurations.

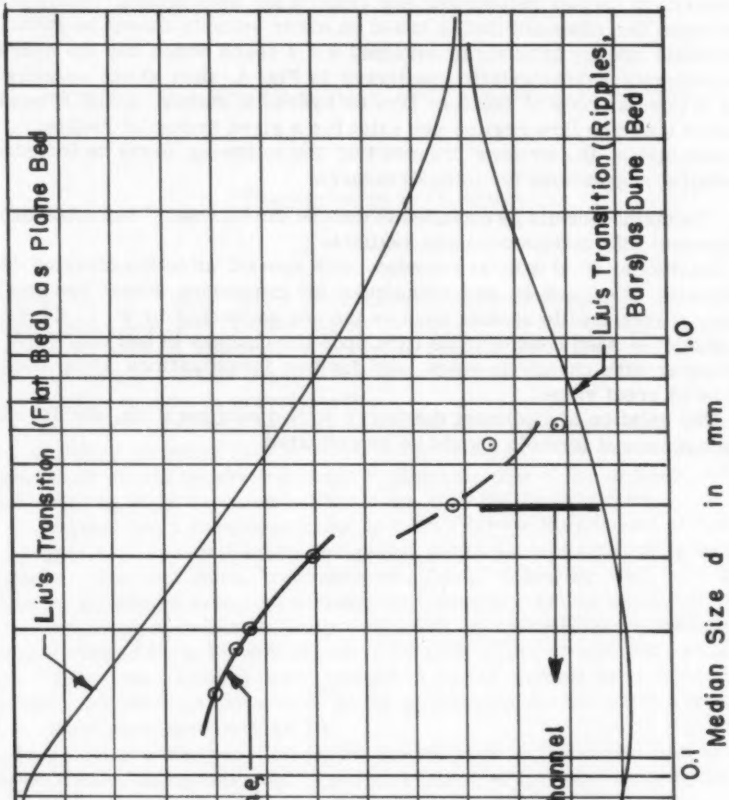


FIG. B.—VARIATION OF C_a WITH d FOR FLOW OVER ALLUVIAL BED

The writers further believe that any criteria for determining regime of flow in true sand bed channels that is based on shear velocity cannot be sufficient. If a constant energy gradient is assumed for a reach which has the hydraulic radius-velocity characteristics indicated in Fig. A, then shear velocity becomes a function only of depth of flow or hydraulic radius, and it is possible that more than one flow regime can exist for a given hydraulic radius.

In conclusion, the writers propose that the following items be included in the authors' suggestions for future research:

1. The method should be extended to include the "antidune" bed configuration as additional information becomes available.
2. Additional field data are needed, with special attention directed to the development of equipment and techniques for measuring actual bed configurations. River profile studies such as the one conducted by W. C. Carey and M. D. Keller[11] shed considerable light upon the changes in bed configurations which occur with change in stage, and further investigations along this line would be of great value.
3. The relation of sediment discharge to bed configuration, and its effect upon the proposed formula should be investigated.

[11] "Systematic Changes in the Beds of Alluvial Rivers," by Walter C. Carey and M. Dean Keller, Proceedings, ASCE, Vol. 83, No. HY 4, August, 1957.

EFFECT OF AQUIFER TURBULENCE ON WELL DRAWDOWN[a]

Discussion by E. G. Kruse

E. G. KRUSE.[1]—The effect of turbulence on head losses in aquifers near pumped wells has been considered by the author. In the West many wells are constructed with an envelope of gravel to prevent sand from moving into the well. In the light of this paper it would be interesting to consider the effects of turbulence on flow in gravel packs.

In the gravel pack both velocity and grain size are larger than in the aquifer, therefore turbulent flow is more likely to exist. The permeability of the gravel pack should be several times higher than that of the aquifer. The head loss resulting from turbulence, therefore, may not be important.

A series of tests have been made by the writer for the purpose of determining proper size and uniformity for gravel packs for use with wells in various aquifers. The test model represented a 30-in. diameter well with a 12-in. diameter screen, a size representative of many irrigation wells. Seven piezometers along a radius in the gravel pack permitted head readings. Discharges ranged from the equivalent of 10 to 100 gal per min per vertical ft of well. Reynolds numbers were computed by the method used by the author. Reynolds numbers in the gravel packs at the well screen varied from 2.3 to 94.0. Most were greater than 10.

To determine whether flow in the gravel pack of the model was in a laminar or transition regime, the Reynolds number at the well screen was calculated. The value of n in the expression $V^n = K \frac{\Delta h}{L}$ was determined according to the author's table. Then using the head readings at two points in the gravel pack, one near the pack-aquifer interface and the other near the well screen, the value of K was calculated for two cases, n as listed under the heading "Conclusions from Analysis of Test Data" and n equal to 1 (assumed laminar flow). The head loss curve in the gravel pack was plotted for each assumed n and compared with the experimental data for intermediate points in the gravel pack.

For Reynolds numbers on the order of 20, the assumed laminar flow curve agrees slightly better with the experimental data than does the transition curve. This is illustrated in Fig. 1 for one test typical of several at this Reynolds number. The agreement of the data with the laminar curve is therefore apparently real rather than due to experimental error. For tests with higher Reynolds numbers at the well screen the experimental data generally plotted between the assumed laminar and transition curves. It may be concluded that some head loss due to turbulence often exists in gravel packs for Reynolds

[a] November, 1959, by Joe L. Mogg.

[1] Agric. Engr., Western Soil and Water Mgt. Research Branch, Agric. Research Service, U. S. Dept. of Agric., Fort Collins, Colo.

numbers greater than 20. The values of velocity exponent listed by the author are probably slightly high due to the factor of safety that the author has included.

The head losses in the model tests were 6 in. or less through properly selected and developed gravel packs. This value would include the component of

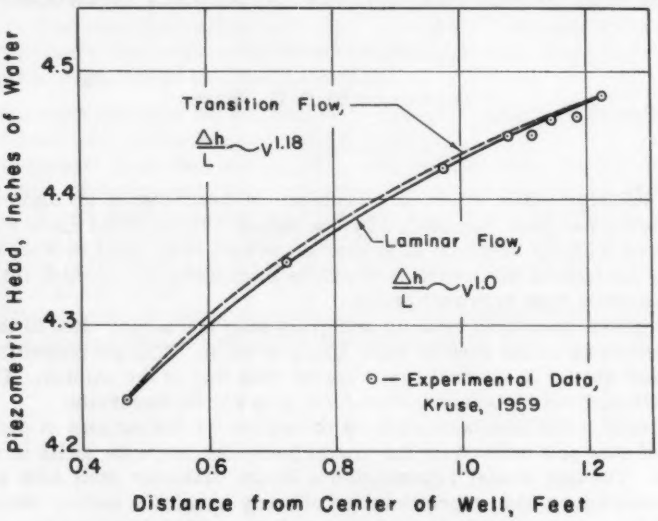


FIG. 1.—HEAD LOSS CURVES FOR ASSUMED LAMINAR AND TRANSITION FLOWS

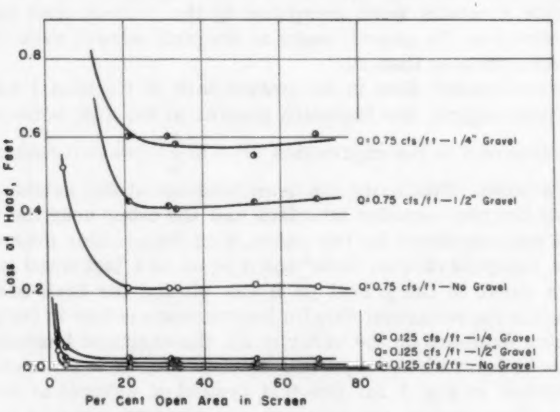


FIG. 2.—HEAD LOSS THROUGH WELL SCREENS

loss due to turbulence. For a pumping lift of 50 ft or more the total head loss through the gravel pack would amount to less than 1% of the total lift and the component due to turbulence could surely be neglected.

Reynolds numbers in the aquifer at the pack-aquifer interface were mostly less than ten and in all but two tests were less than 20. It would seem that head

loss due to turbulence in the aquifer of a large diameter gravel-packed well can also be neglected in computing drawdown in wells.

Experimental data obtained by G. L. Corey, Jr.[2] (Fig. 2) support the author's observation that screens with insufficient open area may cause significant head losses. Corey's tests showed that head losses through the screen decreased as C_s, the percentage of open area, increased. For C_s above some critical value, head losses remained constant. This critical value was dependent on discharge and the size of material surrounding the screen. Corey also found that the head losses through the gravel packs were quite small compared to total losses through the well screens.

2 "Hydraulic Properties of Well Screens," by Gilbert Lee Corey, Jr., Unpublished Master's Thesis, Colorado State Univ., June, 1949.

[illegible]

THE VORTEX CHAMBER AS AN AUTOMATIC FLOW-CONTROL DEVICE[a]

Discussion by John C. Stevens

JOHN C. STEVENS,[1] F. ASCE.—This paper was obviously submitted before Paper 1461 of the author's reference 5 was printed in Transactions as Paper 3004, Vol. 124, 1959, p. 871. In Transactions, the Vortex Number of the authors' signs and symbols has been changed to the Kolf Vortex Number and given the symbol $\underline{\underline{K}}$ in recognition of the original contribution to the science of hydraulics by Richard C. Kolf, M. ASCE. In this discussion the symbol $\underline{\underline{K}}$ is, therefore, used instead of $\underline{\underline{V}}$.

The authors have added to our knowledge regarding vortex flow through horizontal orifices. In particular they have shown that the orifice discharge coefficient C can be determined approximately from the velocity of the water entering the vortex chamber. They have also established the relation between $\underline{\underline{K}}$ and α, the angle of divergence of the water from the horizontal as it leaves the orifice, to be

$$\underline{\underline{K}} = \pi \cos \alpha \qquad (1)$$

Otherwise $\underline{\underline{K}}$ had heretofore been determined from the circulation Γ, the orifice diameter D and the average velocity of water, through the orifice, to be

$$\underline{\underline{K}} = \frac{\Gamma}{D\sqrt{2gH}} \qquad (2)$$

As stated by the authors, the water surface profile computations were "based on a measurement made only one diameter away from the orifice" that is, by substituting D for X in Fig. 1. It was found in the University of Wisconsin experiments that coordinates of this point were a fair index of the hyperbolic water surface profile of a free vortex[2] given by (28) except perhaps near the orifice where viscous forces are in evidence.

The writer was interested in knowing how closely the values of $\underline{\underline{K}}$ by the two methods agreed. From the authors' table has been prepared.

TABLE 1

α^{o}	cos α	$\underline{\underline{K}}$	π cos α	$\underline{\underline{K}}/\pi$ cos
70	.342	.95	1.07	.89
64	.438	1.25	1.38	.91
60	.500	1.70	1.57	1.08
52.5	.609	1.86	1.91	.97
46	.695	2.24	2.18	1.03
49	.656	2.12	2.06	1.03

[a] December, 1959, by R. C. Kolf and P. B. Zielinski.

[1] Cons. Hydr. Engr., Stevens and Thompson, and Leupold and Stevens Instruments, Inc., Portland, Ore.

[2] Transactions, ASCE, Vol. 124, 1959.

The average of the ratios in the last column is 0.985 indicating that the average value of K by either approach was within 1-1/2% of each other, although the maximum variation was 11%. With a more refined method of measuring this angle, which the authors admit was difficult in the Marquette University laboratory, better individual agreements could doubtless be expected.

A comparison of the discharge coefficients versus the Kolf vortex number for both the Marquette and the Wisconsin experiments are shown in Fig. 1.

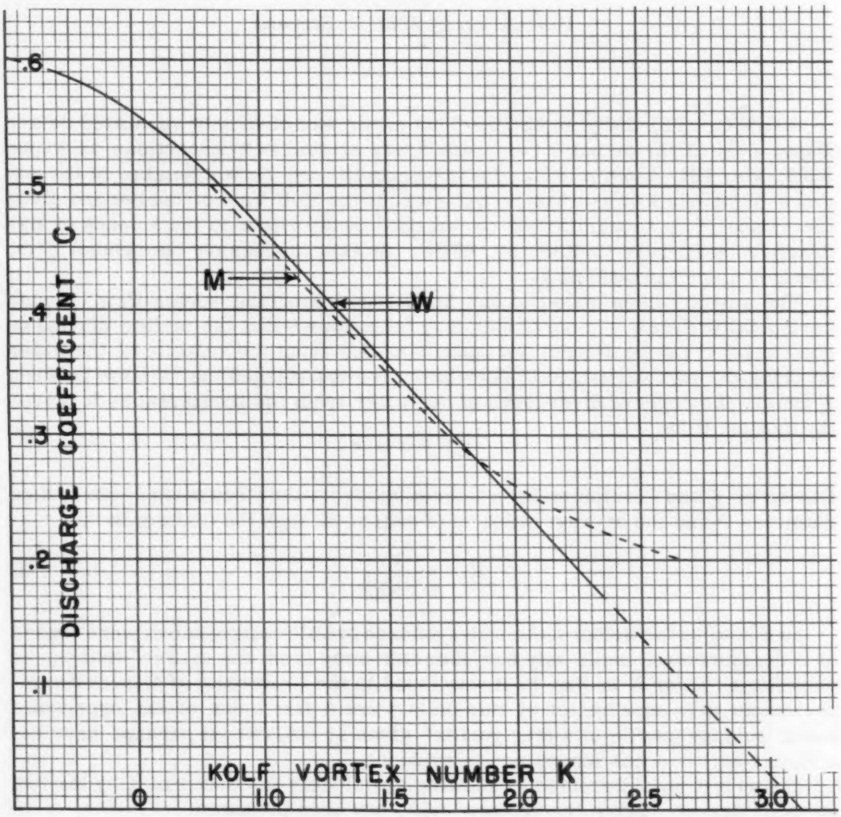


FIG. 1.—COMPARISON OR COEFFICIENTS AND VORTEX NUMBER

Curve M is taken from the authors' Fig. 2 and is defined by experimental data, while Curve W is from the composite curve of the authors' reference 5, the solid portion of which is similarly defined. The deflection toward Grandma's near the bottom of Curve M might disappear if some allowance could be included for the non-hyperbolic portion of the vortex surface profile as water approaches the orifice.

THE SETTLING PROPERTIES OF SUSPENSIONS[a]

Discussions by Lucien M. Brush, Jr. and Hau-Wong Ho, and E. J. Hall

LUCIEN M. BRUSH, Jr.[1] and HAU-WONG HO.[2]—The author deserves considerable praise for his lucid presentation of an interesting subject. The detailed presentation of the continuity equation for a sediment-water suspension in turbulent motion is straightforward and pertinent to the ensuing experiments. The experiment and the interpretations of the results are clearly presented, as are the comments on similarity, and the speculations as to the application of the multiple-depth analysis to sediment problems involving open-channel flow and sediment removal. In addition to the interpretations made by the author concerning the experiment, several other interpretations exist which the writers feel are worthy of comment. Because the core of the author's paper is based on the so-called measurement of $\bar{\bar{w}}$, the local mean settling velocity, this discussion will be directed toward the experimental and interpretive aspects of the investigation.

In presenting the continuity equation for sediment suspensions, the author states that turbulence will appear explicitly through the diffusion terms, whereas flocculation or hindered settling due to turbulence will occur implicitly in the fall-velocity term. Nevertheless, the results obtained by the author for the multiple-depth analysis are interpreted to be due to flocculation alone. Thus, any effect due to turbulence, either implicitly or explicitly, is discounted. There seems to be little justification in ignoring turbulence, because it is most certainly present, at least initially, if the procedure outlined by the author is followed in making the analysis. Turbulence enters the system as a result of "thoroughly mixing" the sample (to obtain a uniform dispersion) and as a result of pouring the sample into the measuring cylinder. Obviously, the intensity of the disturbance caused by this experimental procedure will decrease with time due to the viscous effects of the fluid, but the net outcome of the introduction of decaying turbulence will be to generate curves similar to those obtained by the author in Fig. 8—with or without the presence of flocculation.

Although it is difficult to measure the scale and intensity of the introduced turbulence without running rather elaborate experiments, it is possible to make a qualitative estimate of the amount of time that the turbulence effect would be of importance in an experiment similar to that made by the author. The writers placed fluid droplets with a specific gravity slightly greater than one in a container filled with water and thoroughly mixed the liquid to obtain a dispersed suspension. The contents of this container were then quickly poured into a cylinder which had dimensions similar to the one described by the author. Observations were then made of the turbulence activity by noting the direction of

[a] December, 1959, by Ronald T. Mc Laughlin, Jr.

[1] Research Engr., Iowa Inst. of Hydr. Research, and Asst. Prof., Dept. of Mechanics and Hydr.; State Univ. of Iowa, Iowa City, Iowa.

[2] Research Assoc., Iowa Inst. of Hydr. Research, State Univ. of Iowa, Iowa City, Iowa.

motion of the fluid droplets. Three minutes after pouring, it was noted that about 30% to 40% of the fluid droplets were moving upward against gravity. After 7 min, less than 5% of the particles were moving upward, but the effect was still noticeable. Thus, there seems to be reasonable evidence that some turbulence effect would exist for about 10 min after pouring the sample. It would follow therefore, that some doubt might be raised as to the applicability of the author's Eq. 17

$$\frac{\partial \phi}{\partial \phi} + \frac{\partial(\bar{w}\ \phi)}{\partial z} = 0 \qquad (1)$$

in computing the local mean settling velocity, because a diffusion term should be included in the equation. The appropriate equation would be,

$$\frac{\partial \phi}{\partial t} + \frac{\partial(\bar{w}\ \phi)}{\partial z} - \frac{\partial\left(e_z \frac{\partial \phi}{\partial z}\right)}{\partial z} = 0 \qquad (2)$$

Integration of this equation with respect to z will yield,

$$-\int_0^D \frac{\partial \phi}{\partial t}\ dz = \bar{w}\ \phi - e_z \frac{\partial \phi}{\partial z} \qquad (3)$$

instead of Eq. 18, as given by the author. A numerical solution of the integral may be obtained by determining the slope, for a particular depth, of a plot of the area under the concentration curve against time, but the numerical solution will not discriminate between the author's Eq. 18 and Eq. 3. Thus, if the numerical result is divided by the concentration, an apparent fall velocity—here called $\bar{w}_a$—will be determined which will approach the author's $\bar{w}$ only in the absence of turbulence or as the turbulence decays sufficiently to cause the diffusion term to be insignificant. Assurance that the diffusion term of Eq. 3 is important initially, may be obtained by considering that in order to obtain an initially uniform dispersion of sediment, the following equation must apply

$$\frac{\partial(\bar{w}\ \phi)}{\partial z} - \frac{\partial\left(e_z \frac{\partial \phi}{\partial z}\right)}{\partial z} = 0 \qquad (4)$$

wherein by definition the two terms must be equal. As the turbulence decays, the diffusion term will become less important and will cause $\bar{w}_a$ to rise to a peak and then decrease as a function of time in much the same manner as reported by the author (Fig. 8) for flocculation. It follows therefore, that the effect of decaying turbulence on computations of the local mean settling velocity cannot be readily distinguished from the effect caused by flocculation without some other independent check. It is also interesting to note that the peak of the curve shown by the author (Fig. 8) occurs about 10 min to 12 min after the initial pouring which is the same order of magnitude as the time necessary for the turbulence effect to become negligible, as estimated by the writers.

Aside from the explicit effect of turbulence, an implicit effect may also occur; that is, the settling of individual grams will be hindered—at a decreasing rate after pouring—as a result of turbulence. Decaying, hindered settling would also tend to cause a peak to occur on a fall velocity - time graph. In order to visualize these various effects, a schematic illustration, showing plots of fall velocity versus time for a particular sampling depth is given in Fig. A.

Each of the reported turbulence effects—decaying turbulence or decaying hindrance of settling—would tend to cause the lines of constant Z/t to slope towards the Z axis of a concentration - depth graph (Fig. 7) and therefore behave similar to a suspension in which flocculation occurs. Because of this and the previously stated arguments, there is considerable doubt in the minds of

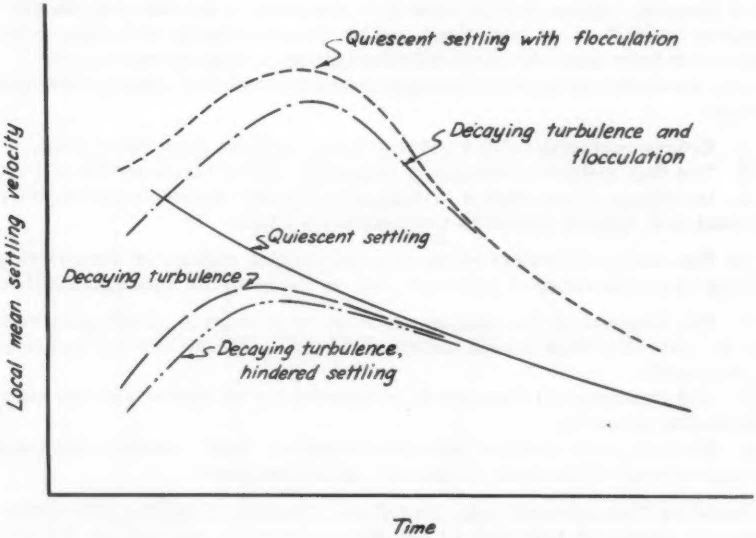


FIG. A.—EFFECT OF TURBULENCE, HINDERED SETTLING, AND FLOCCULATION ON COMPUTATIONS OF THE LOCAL MEAN SETTLING VELOCITY.

the writers as to whether the author's experiment isolates the effect of flocculation. Presumably, at great depths, turbulence would decay before an appreciable change in the concentration gradient would occur, thereby reducing the importance of including a diffusion term in the continuity equation. Perhaps this would enable the separation of flocculation from the other effects, but in order to establish the proper depths for sampling, further experimentation would be required.

E. J. HALL.[3]—This paper is an important addition to the literature on sedimentation and serves to narrow the gap between the theoretical approach to the process and practical design.

It is common to all the theoretical approaches from the time of Hazen that basic assumptions on the hydraulic flow in a tank and the settling characteristics of the suspension have to be made which are necessarily simplified to the point that practical application of the resultant theory is of doubtful value.

The settling characteristics of the suspension, in particular, is a factor which has not received the attention which it warrants and the common assumptions made that the particles have a uniform settling rate, or that they settle with uniform acceleration, is far removed from the truth, particularly where sewage is concerned. The graphs in this paper clearly illustrate this point.

[3] Chf. Planning Engr., City Engr.'s Dept., Johannesburg, S. Africa.

The multiple depth pipette analysis suggested, deals adequately with this factor, but its application is likely to be difficult or at the least laborious in practice.

In the search for a simple practical method of determining the settling characteristics of a sewage, the writer has made a close study of the process as it takes place in the laboratory and a brief report of the developments may be of interest. Observations show that even after a sewage sample has been standing for an hour or two, flocs can be seen settling at such high rates that they settle from near the surface to the bottom in a few minutes.

It is presumed from these observations that settling of sewage proceeds as follows:

1. Coarse material settles out with a few minutes quiescence only.
2. The fine material remains in suspension slowly forming flocs.
3. Once these flocs reach a critical size they commence to settle, and their removal in a shallow vessel is comparatively rapid.

On this basis, the selection of an experimental method of determining the settling characteristics of a sewage, can be made on the assumptions that:

a. The removal of the coarse material in sewage is easily achieved and may be ignored in tank design (except in as far as sludge storage and removal is concerned).

b. The fine material remains in suspension for an appreciable period while flocculation proceeds.

c. Once flocs of a critical size are produced, their removal, compared to the times required for their formation, is instantaneous.

Based on these assumptions, therefore, samples taken from the surface of quiescent sewage at intervals as settlement proceeds will reflect the settling characteristics of that sewage. In the application of the method to practice, it may be added that as far as the instantaneous removal of flocs, once formed, is concerned, the assumption can be justified if the overflow rate of the tank under consideration is less than the settling velocity of the floc.

The apparatus used by the writer to determine the settling characteristics of a sewage, consists of a drum 11 in. in diameter and 17 in. deep to which a small floating draw off, of 1/4 in. bore tube suspended on two corks, is fitted. A batch of sewage is added and samples removed from the surface at intervals. The samples are analysed either for suspended solids or preferably for P.V. (4 hr O.A.) as this is of more significance in works design. The experiment is repeated many times and the results averaged and represented graphically as quality of effluent (or surface liquor) against time. The method of analysis is well adapted to demonstrate the value of mechanical flocculation, as the batch after adding to a test vessel may be agitated in the vessel for a given time, at given paddle speeds, after which the paddle mechanism is removed, the floating draw off replaced and samples taken in the usual way. The use of two identical vessels simultaneously enables a direct comparison to be made between pure quiescence and flocculation followed by quiescence. Fig. 1 shows a typical settling characteristics analysis curve of a domestic sewage and the characteristics of the same sewage when agitated by an oscillating paddle for 30 min at a maximum paddle velocity of 4.5 ips.

It would appear, on first sight, easy to determine the efficiency of a sedimentation tank by relating its effluent quality to time of quiescence (or effective detention period) by reference to the settling characteristics curve. Variations in settling characteristics and sewage strength during the day are such,

however, that a reliable measure can only be obtained by averaging large numbers of readings. A further factor is that flocculation does not proceed at a fixed rate but is greatly speeded up by slow mechanical agitation. Consequently, it must occur in varying degrees in all sedimentation tanks. For a graph to which direct reference could be made, it would, therefore, be necessary to either:

1. Determine the extent and duration of the agitation occurring in the tank and duplicate this in the vessel used for analysis or

2. Determine the optimum conditions for flocculation in the vessel and thus obtain a curve which could be assumed to represent the optimum and ideal condition.

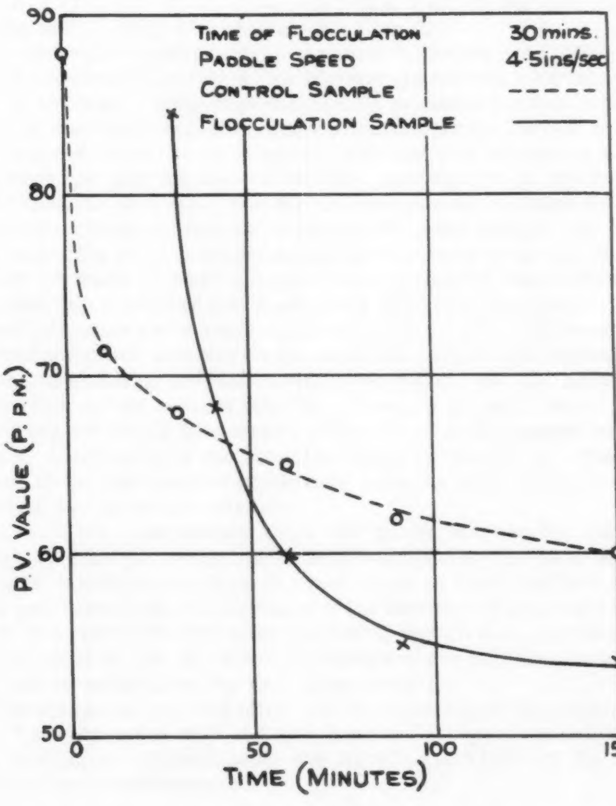


FIG. 1.—SETTLING CHARACTERISTIC ANALYSIS CURVE

It is obvious that even with the simplified approach outlined previously, the prediction of tank performance from basic data bristles with difficulties in which unknown factors abound. It is to be hoped that further research such as that described in this paper will be carried out to bring more light to this fascinating problem.

[illegible]

TIDAL CHARACTERISTICS FROM HARMONIC CONSTANTS[a]

Discussion by Remig A. Papp

REMIG A. PAPP,[1] M. ASCE.—The paper fills a gap in our information about tides and the author should be congratulated for it.

I would like to add supplementary comment to the section: Meteorological Effects. In addition to the winds, the barometric pressure can also influence the height of the tide. After high barometric pressure the tide will be lower than computed and predicted; after a low pressure period the tide will be higher. The difference could be quite appreciable and may amount to 6 in. to 8 in.

This writer has participated in the port construction of the Port of Dunquerke, France, in 1931-32. For the construction of the Jetee Est, 40 reinforced concrete caissons, precast in the inside of the harbor, had to be towed out to the end of the jetty in construction. Due to the draft of the caissons this towing could be made only twice a month during highest tide and every major deviation from the predicted tide level could have been critical.

The field office has had a registering barometer and the differences between the predicted and actual tide elevations were plotted under the barometric pressure curve. By plotting the plus deviation downward and the minus deviation upward, the two curves became similar. The tide curve, however, had had a pronounced lag behind the barometric curve which might amount to 10, 12, or more hours. A difficulty is that the differences or deviations in the tide could be observed at the high and low points only since the other points are generally not computed, having no importance.

The notes of the observations were lost during the war but they were not long enough to draw too many conclusions from them. It would be, however, quite easy to establish this type of observation in a few harbors with a high tide range and to evaluate the influence of the barometric pressure on tide elevation. It is conceivable that after collecting enough data, a relation between the angle of slope of the barometric pressure curve and the deviation in tide elevation can be established for each local condition.

In narrow estuaries the wind effect and the influence of the barometric pressure may have the same sign and may have to be superimposed or they may have opposite signs. In such cases the direction and force of the wind should also be taken into consideration.

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[a] December, 1959, by Bernard D. Zetler.
[1] Assoc., Hazen and Sawyer Engrs., New York.

[illegible]

[illegible]

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[illegible]

[illegible]

[illegible]

EARLY HISTORY OF HYDROMETRY IN THE UNITED STATES[a]

Discussions by William G. Hoyt and E. Shaw Cole

WILLIAM G. HOLT,[1] M. ASCE.—Mr. Kolupaila uses the beginning of World War I (1914) as the end of the pioneer stage in the field of water measurement known as stream gaging. In order to complete his listing of pioneers in this field it seems appropriate to include other engineers who added to the knowledge of the field in those early days. The writer refers to the following who were district engineers of the United States Geological Survey, Dept. of the Interior (USGS), in 1918 and who were not otherwise mentioned in the paper:

C. E. Ellsworth, in Texas; A. L. Baldwin, in Idaho; F. F. Hemshaw, in Oregon; O. W. Hartwell, in New England and New Jersey; A. H. Horton, in the Ohio River Basin and who initiated the first nationwide system of collecting estimates of developed and potential water power; William Lamb, in Montana; H. D. McGlashan, in California; G. L. Parker, in Washington state and later chief hydraulic engineer (1939 to 1946); C. G. Paulsen, in the south Atlantic states and later chief hydraulic engineer (1946 to 1957); A. B. Purton in the Great Basin; R. C. Rice, in Kansas; and G. C. Stevens, in the Middle Atlantic region.

Mr. Kolupaila suggests that those who measure water might well be called "hydrometrists." Perhaps the art of stream gaging has progressed to the point where such a designation would be appropriate. The engineers referred to in the paper or listed herein were far more than hydrometrists. Had they been only what the definition implies, the Water Resources Division of the USGS would not be the authoritative agency on national water resources it is today.

Prior to World War I, the actual time spent measuring streams took but a small part of each 24 hr, a fact to which the writer can personally attest as one of the pioneers listed by Mr. Kolupaila. When not traveling by train, livery, or on horseback, stream gagers were carpenters building structures; laborers chopping ice; public relation experts keeping farmers, ranchers, and others happy and reading gages twice a day for a maximum of $5 per month; computing gage heights, preparing rating curves; making monthly reports to Washington; and above all, exercising expert salesmanship by convincing state and city officials, heads of irrigation districts and hydropower companies, and any others who would listen, of the need for streamflow records. Without the financial support thus obtained and the enthusiasm engendered, the work would not have been maintained and certainly would not have continued at an ever-expanding rate to the present time. With but small staffs and large regions to cover, those in charge of the few district offices managed to maintain some 1,000 gaging stations at a total cost of approximately $300,000, of which nearly one-half was provided by state, public, and private agencies.

In some quarters these early engineers have been criticized for not taking time to make a myriad of other observations deemed today as essential for

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[a] January, 1960, by Steponas Kolupaila.

[1] Cons., Senate Select Committee on Natl. Water Resources, Washington, D. C.

proper water management and also for not thinking more profoundly about the hydrology of the river basins and the hydraulics and morphology of the streams. In general, it can be said that they were largely stream gagers, pure and simple, but the records they recollected by sticking to their respective assignments were the beginning of a storehouse of streamflow information that now covers some 200,000 station-yr of records, with annual expenditure by the USGS for all phases of water information now running about $25,000,000, of which cooperating bodies—local, state, and federal—still contribute about one-half.

Unfortunately, at the beginning of the century there was no immediate need (and the future need was not adequately recognized) for information about ground water, the quality of water, the movement of sediment and other hydrologic data now considered essential in the management of water resources. Even as extensive as all the information on water resources is today, it is still far from sufficient to meet present-day needs if information now being collected by the Senate Select Committee on National Water Resources, of which Senator Robert S. Kerr of Oklahoma is chairman, is at all indicative.

The writer is having an opportunity to analyze the extensive reports being received from the governors of the 50 states in response to a request from Senator Kerr for information as to present water problems and those anticipated between now and 1980, together with recommendations as to what should be done toward their solution. Also, being reviewed is the testimony of some 750 witnesses at 23 public hearings held throughout the country and numerous reports from federal agencies, consultants, and others.

Although the writer is somewhat familiar with the availability of hydrologic data and deficiencies therein,[2] he has been truly amazed with the apparent need disclosed in these sources for more hydrologic and meteorologic information of all kinds if solutions to the water problems of today and those anticipated in 1980 are to be met. The extent to which this information is collected, analyzed, interpreted, and disseminated will have a profound effect on the rapidity and efficiency with which present and anticipated water problems will be solved.

E. SHAW COLE,[3] F. ASCE.—There is some confusion in the account of the pitot tube as used to measure the flow in pipes. A number of published papers on the subject have sought to establish that there are two types, the simple and the combined. The simple pitot tube is essentially a bent tube with the nozzle or orifice facing the flow. The total pressure against the orifice is a combination of velocity head plus static head and to measure velocity in a pipe under pressure the static pressure must be measured by a wall piezometer. This is the type of tube used by Mills and Freeman.

The combined type of pitot tube included a second tube that measured the static pressure. This was generally a long tube that was suited to open channel flow measurements. The Pitometer was a special type of combined tube that used two identical orifices 180° apart, one facing upstream and the other downstream. This was developed and patented by Edward S. Cole and given the coined name that included Henri Pitot's name. Its special features were its compact size that permitted it to be installed easily in a pipe under pressure, thus making it portable, its accuracy and its reversibility which provided a simple means for checking zero.

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[2] "Water Facts For the Nation's Future," by Langbein and Hoyt, Ronald Press Co., New York, 1959.

[3] Pres., The Pitometer Assocs., New York, N. Y.

Two papers by Edward S. Cole,[4,5] one by C. W. Hubbard,[6] and the Power Test Code for 1949,[7] are important references for any one interested in the development or use of the pitometer.

4 "Pitot Tube Practice," by Edward S. Cole, Transactions, ASME, Vol. 57, 1935, p. 281.

5 "Pitot Tubes in Larger Pipes," by Edward S. Cole and E. Shaw Cole, Transactions, ASME, Vol. 61, 1939, p. 465.

6 "Investigation of Errors of Pitot Tubes," by C. W. Hubbard, Transactions, ASME, August, 1939.

7 "Power Test Code for Hydraulic Prime Movers," 1949, pp. 135-165.

ERRATA

Journal of the Hydraulics Division

Proceedings of the American Society of Civil Engineers

February, 1960

p. 41. The letter a at the end of Eq. 3 should be made a superscript.

p. 41. In the tabulation near the bottom of the page the less than sign should be moved closer to the 1/2 in the first line.

p. 47. In the equation following Eq. 11 the term 2^1 should be changed to 2_1.

p. 51. In line 11 the words "and the stage of the floodplain" should be changed to "and the state of the flood plain."

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST10) which indicates that the paper is contained in the tenth issue of the Journal of the Structural Division during 1959.

VOLUME 85 (1959)

JUNE: [illegible] 2054(CP1), [illegible] 2057(HY6), [illegible] 2061(PO3), 2062(SM3), 2063(SM3), [illegible] 2070(WW2), [illegible] 2077(SM3)^c, [illegible]

JULY: [illegible] 2081(HY7), [illegible] 2084(HY7), [illegible] 2086(SA4), [illegible] 2097(HY7), [illegible] 2100(AT3), 2101(AT3), [illegible] 2107(AT3), [illegible] 2110(AT3), 2111(AT3), [illegible] 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), [illegible] 2124(AT3).

AUGUST: 2126(HY8), 2127(HY8), [illegible] 2130(PO4), 2131(PO4), [illegible] 2135(SM4), 2136(SM4), 2137(SM4), [illegible]

SEPTEMBER: 2141(CO2), 2142(CO2), [illegible] 2150(HY9), 2151(IR3), [illegible]

OCTOBER: [illegible]

NOVEMBER: [illegible]

DECEMBER: [illegible]

VOLUME 86 (1960)

JANUARY: [illegible]

FEBRUARY: [illegible]

MARCH: [illegible]

APRIL: [illegible]

MAY: [illegible]

JUNE: [illegible]

c. Discussion of several papers, grouped by divisions.

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PART 2

JUNE 1960 — 19
VOLUME 86

NO. HY 6
PART 2

Your attention is invited

NEWS OF THE HYDRAULICS DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE HYDRAULICS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

JUNE 1960

THE PURPOSE OF THE HYDRAULICS DIVISION

"The advancement and dissemination of scientific and engineering knowledge in all branches of hydraulics, hydrology, hydraulic engineering and water resources. In particular this shall embrace meteorology and hydrology as the sciences dealing with the occurrence of water in the atmosphere, on the earth surface and in the ground, fluid mechanics for the understanding of all flow phenomena, applied hydraulics for the design and planning of hydraulic structures and of comprehensive systems, and those social, economic and administrative aspects basic to the conservation and utilization of water as an essential natural resource."

All committees and task forces together with their members are listed in the Official Register for 1960.

Committee Meetings

Division Technical Committees are expected to be extremely active in the future. Several meetings have been held and others are being planned. Members of the Hydraulics Division can look forward to many interesting programs as a result of this activity. Brief notes on the committee meetings will acquaint you with some of the activities.

Committee on Sedimentation

This committee met in Chicago on April 9 and 10, with Chairman E. J. Carlson and members A. P. Gildea, P. C. Benedict, P. S. Eagleson and R. B. Banks attending. The committee firmed up the program for the session at the Seattle Conference (see Seattle program in this issue of the Newsletter), made plans to complete the Sedimentation Manual at an early date, laid plans for sessions in future conventions and reviewed the activities of existing task forces. Chairman Carlson will plan a session for the April 10-14, 1961

Note.—No. 1960-19 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 2, June, 1960.

Phoenix Convention under the theme "Scour of Cohesive Materials," and member Eagleson will plan a session for the August 16-18, 1961 Urbana Hydraulics Conference under the theme "Reservoir Sedimentation." The committee's future plans include setting up a task force to prepare a bibliography of translations and foreign literature on various aspects of sedimentation.

Committee on Hydraulic Structures

This committee met on March 11 during the New Orleans Convention with Chairman A. J. Peterka and members H. K. Pratt, R. A. Elder, M. J. Webster and G. E. Hands attending. The activities of the Committee and the eight task forces were reviewed in detail and plans made for activities including sessions for future conventions. The aims, accomplishments, status, objectives, progress, and action for each task force were considered individually and in detail. Extensive future activity is expected. Many of the task forces will plan and prepare material for future sessions. The program for the Seattle Conference has been completed (see Seattle program in this issue of the Newsletter). A session for the Boston Convention (October 10-14) under the theme "Hydraulics of Bridges" is being arranged by G. E. Hands. A session on "Flow in Large Conduits" to be arranged by R. A. Elder is contemplated for the Phoenix Convention, April 10-14, 1961. Sessions with themes of "Hydraulics of Culverts," "Energy Dissipators," and "Hydraulics of Bridges," are to be planned for the Urbana Hydraulics Conference, New York Convention, and Houston Convention. The committee proposes to set up a new task force on "Hydraulics of Navigation Locks" to gather and present information on recent progress and changes in design practices.

Committees on Hydrometeorology, Surface Water Hydrology, and Flood Control

These committees held a joint meeting April 21 and 22 in Chicago, with Chairman R. K. Linsley, and members W. E. Hiatt, A. T. Lenz, G. E. Harbeck, and A. O. Waananen of the Hydrometeorology Committee, Chairman H. A. Foster, and members W. C. Ackermann, C. C. McDonald and S. W. Jens of the Surface Water Hydrology Committee, Chairman A. T. Lenz of the Flood Control Committee, and H. S. Riesbol, Water Resources Coordinator attending. The committees planned and coordinated their program of work for the year. Objectives and scope of work were agreed upon and task assignments were made to committee members. Sessions for the Society Convention in Phoenix, Arizona, on April 10-14, 1961, and the Hydraulics Division Conference in Urbana, Illinois, on August 16-18, 1961, were planned. Task group objectives and assignments were reviewed.

Hydrologic Materials Preserved

Because of the increased interest and activities of the Hydraulics Division in Water Resources, a recent article from the February 1960 issue of "Western Water News" is quoted below. The new Water Resources Center Archives is unique in the United States and should be of interest and value to ASCE members interested in water resources.

"ARCHIVES PROGRAM PURSUES HYDROLOGIC MATERIALS

U. C. Center Seeks Cooperation of Water Agencies

As a part of a state-wide University of California research program on water resources, the Water Resources Center Archives on the Berkeley Campus is gathering materials and data in the water resources field. The collection will include all aspects of water: water as a natural resource and its utilization; irrigation; flood control; municipal and industrial water uses and problems; water rights; and water development projects. The emphasis is on material relating to the State of California.

Besides actually acquiring material, the Archives is interested in locating and making biographical listings of important collections which remain in public or private agencies and in engineering offices. A comprehensive catalog will thus be developed for the use of researchers, describing and giving the location of existing unique hydrologic data in the State.

Cooperation Sought

Because the materials found to be the most valuable are not available in the sense that they can be purchased such as most libraries purchase books, the Archives relies upon the cooperation of interested individuals and agencies for augmenting its collection. Engineers' reports and studies (unpublished materials, manuscripts, etc.), agency reports, pamphlets, hearings, legislation, specifications, maps, scrapbooks, newspaper clippings, speeches, campaign material, and books pertaining to water: all are being sought. Interests include both current materials and the historical aspects.

The entire collections of several men prominent in the water resources engineering fields are now housed in the Archives: among these important gift collections are those of Frank Adams, Bernard A. Etcheverry, Charles Gilman Hyde and Baldwin M. Woods.

Seven reports have been prepared and published by the Archives, all of them available to the interested public:

1. The Etcheverry Collection in Water Resources Archives (June, 1958).
2. Theses on Engineering, Economic, Social and Legal Aspects of Water (October, 1958).
3. Watershed Management Research Data, U. S. Department of Agriculture, Forest Service, California Forest and Range Experiment Station, Berkeley and Glendora, California (February, 1959).
4. Publications and Reports of Charles Gilman Hyde (July, 1959).
5. Water Pollution Data, Regional Water Pollution Control Boards, State of California (July, 1959).
6. Bachelor of Science Theses on Water Resources Engineering, University of California, Berkeley (August, 1959).
7. Theses on Water Resources, Stanford University, California Institute of Technology, and University of Southern California (August, 1959).

Agencies dealing with water in any of its forms or uses are requested to add the Archives to their mailing lists. Persons having materials which have served their purposes are urged to consider the Archives a repository for the filing of these materials.

The Archives serves researchers in the University community as well as the interested public. The activity is under the direction of Professor J. W. Johnson, Director of Hydraulic Laboratories, University of California, Berkeley. Gerald J. Giefer is the Librarian directly in charge of the Archives.

The mailing address of the Archives is: Water Resources Center Archives, Room 5, Mechanics Building, University of California, Berkeley 4, California."

NINTH HYDRAULICS DIVISION CONFERENCE, SEATTLE, WASHINGTON
August 17-19, 1960

Include this conference in your vacation plans. The University of Washington and the Seattle Section are cohosts and the setting will be the University of Washington campus.

The following program is indicative of the many interesting activities being planned.

General Information

Housing and Dining Facilities — Dormitory accommodations will be available at the University. Many hotels and motels are located within a five mile radius of the campus, and information concerning these will be sent from the local planning committee upon request. University dining facilities will be available for all meals.

Social Events

Pre-Conference Skagit River Tour, August 16 — This is a guided tour through the Seattle City Light integrated three-dam hydroelectric project, located in the northern Cascade Mountains near the Canadian border. The all-day trip combines technical interests with rugged mountain scenery, and includes rides by bus, boat, and inclined railway and a lunch served at the project.

Salmon Barbecue, August 17 — Like the Skagit trip, this is also planned as a family event. Scheduled for the evening of the first day of the formal conference, it provides a time for informal socializing on a boat trip through the Chittenden locks and across salt water Puget Sound, followed by a typical regional salmon barbecue served by the water.

Evening Banquet, August 18 — This traditional event will be the final formal social event of the conference.

Ladies' and Children's Programs — A program including teas, tours, and shopping is being planned for the ladies. Activities for younger members of the family are also being set up.

Technical Program

Session on Hydraulic Structures:

Presiding: M. L. Dickinson, Vice Chairman, Executive Committee and A. J. Peterka, Chairman, Hydraulic Structures Committee.

Effects of Cavitation and Negative Pressures on the Passage of Fish in Hydraulic Turbines
Glen Von Gunten, Chief, Planning and Reports Branch, Corps of Engineers, Walla Walla, Washington

Problems Associated with Water Temperature and Fish Passage
Milo C. Bell, Professor, College of Fisheries, University of Washington, Seattle, Washington

Fish Protective Facilities at Tracy Pumping Plant, Central Valley Project, California
T. J. Rhone, Bureau of Reclamation, Hydraulic Laboratory Branch, Denver, Colorado; Daniel W. Bates, Fishery Research Biologist, Bureau of Commercial Fisheries, Portland, Oregon.

Chief Joseph Dam Stilling Basin—Damage and Analysis of Contributing Factors
Robert H. Gedney, Chief, Planning Section, Corps of Engineers, Seattle, Washington.

Session on Tidal Hydraulics:

Presiding: Eugene P. Fortson, Jr., Member, Executive Committee and Haywood G. Dewey, Jr., Chairman, Tidal Hydraulics Committee.

Prototype Measurements — Columbia River Estuary
John B. Lockett, U. S. Army Engineer Division, North Pacific; Harold A. Kidby, U. S. Army Engineer District, Portland, Oregon.

Detailed Current Studies in the Puget Sound Region
John Dermody and Dr. Maurice Rattray, Department of Oceanography, University of Washington, Seattle, Washington.

A Calibration Method for a Hydraulic Model Subjected to Two Tidal Entrances
L. B. D. Terhune and N. E. J. Boston, Fisheries Research Board of Canada, Pacific Oceanographic Group, Nanimo, B. C.

Session on Hydrology:

Presiding: H. S. Riesbol, Water Resources Coordinator, and Franklin Snyder, Member, Hydrology Committee.

Radar Observations of Topographic Effects on Precipitation in the Pacific Northwest
Herbert Kershaw, Jr., Wilfred G. Jensby, and Professor Fred W. Decker, Science Research Institute, Oregon State College, Corvallis, Oregon.

Errors in Measurement of Reservoir Water Levels
Walter J. Parsons, Jr., Corps of Engineers, Sacramento, California

Fluctuations of Annual River Flows
Dr. Vujica M. Yedvjevich, U. S. Geological Survey, Washington, D. C.

Effect of Time and Space Distribution of Rainfall on the Runoff Hydrograph
J. Amorocho and Professor G. T. Orlob, Department of Civil Engineering, University of California, Berkeley, California.

Session on Flood Control:

Presiding: C. E. Kindsvater, Past Chairman, Executive Committee, and Francis G. Christian, Member, Flood Control Committee.

Development of Spillway Design Flood for Bruces Eddy Dam, North Fork Clearwater River, Idaho
Melvin J. Ord and Robert Conway, Corps of Engineers, Walla Walla District, Walla Walla, Washington.

Seasonal Runoff Forecasting of the Columbia River by the Coastal Flow Index Method
David M. Rockwood, Corps of Engineers, North Pacific Division, Portland, Oregon.

Flood Control Economics, Lower Columbia River
Kenneth Case, Corps of Engineers, Portland District, Portland, Oregon.

The Hydrology of Howard A. Hanson Dam and Reservoir
Norman J. MacDonald, Corps of Engineers, Seattle District, Seattle, Washington.

Session on Hydromechanics:

Presiding: Dr. A. T. Ippen, Chairman, Executive Committee, and Dr. Norman H. Brooks, Member, Hydromechanics Committee.

Report of Task Force on Aearted Flow in Open Channels
Frank B. Campbell (Chairman of Task Force), Chief, Hydraulic Analysis Branch, Waterways Experiment Station, Vicksburg, Mississippi.

Symposium on Mechanics of Ground Water Flow
Drainage Wells in Two-Layered Aquifer
C. E. Jacob, Groundwater Consultant, Los Angeles, California

Nonsteady Flow Toward Wells Partially Penetrating a Water-Table Aquifer
Mahdi S. Hantush, New Mexico Institute of Mining and Technology, Socorro, New Mexico.

Salt Intrusion into Coastal Aquifers
Professor Harold R. Henry, Michigan State University, East Lansing, Michigan

Session on Sedimentation:

Presiding: H. M. Martin, Secretary, Executive Committee and Paul Benedict, Member, Sedimentation Committee.

Formulas for Total Sediment Discharge of Alluvial Streams
Dr. Vito A. Vanoni, California Institute of Technology, Pasadena, California.

Transportation of Materials by Fluids in Pipe
Professor A. R. Chamberlain; N. Yotsukura, Research Assistant; S. S. Karaki, Assistant Professor; Dr. M. L. Albertson, Colorado State University, Fort Collins, Colorado.

Stream Control Planning and Structures
D. C. Bondurant, Hydraulic Engineer, Corps of Engineers, Missouri River Division, Omaha, Nebraska.

Legal Aspects of Sedimentation
C. E. Busby, Water Program Specialist, Soil Conservation Service, Berkeley, California.

Other Meetings

Don't Forget the ASCE Boston Convention, October 10-14, 1960, which will include sessions on Tidal Hydraulics, Hydrology, Hydromechanics, Sedimentation, and Hydraulic Structures. More on this meeting in the August Newsletter.

Geological Society of America Annual Meeting

(This is a repeat from the April Newsletter)

The Division on Engineering Geology of GSA will have a symposium on ground water in Denver, Colorado, on October 30 and 31, 1960, along with special papers on related subjects. A field trip to major projects and points of interest to engineering geologists in the Rockies west of Denver is planned for Saturday, October 29. It is expected that the new Water Resources Committees of the Hydraulics Division will find many common interests with the Division on Engineering Geology of GSA.

Officers for the Division on Engineering Geology include three ASCE members and are:

Chairman, Safford C. Happ, AEC, Grand Junction, Colorado
Vice Chairman, George A. Kiersch (F. ASCE), Southern Pacific Company, Oakland, California
Secretary, Thomas F. Thompson (Aff. ASCE), San Francisco, California
Counsellor, George M. Schwartz, University of Minnesota, Minneapolis, Minnesota
Robert F. Legget (F. ASCE), National Research Council of Canada

For Your Calendar

ASCE Meetings

August 17-19, 1960	Hyrdraulics Division Conference, Seattle
October 10-14, 1960	ASCE, Boston Convention

April 10-14, 1961	ASCE, Phoenix Convention
August 16-18, 1961	Hydraulics Division Conference, Urbana, Illinois
October 16-20, 1961	ASCE, New York Convention
February 1962	ASCE, Houston Convention
May 1962	ASCE, Omaha Convention
October 15-19, 1962	ASCE, Detroit Convention

Non-ASCE Meetings

September 1960	Hydraulic Turbine Research Symposium, IAHR, Nice, France
October 29-November 2, 1960	Annual Meeting Geological Society of America, Division of Engineering Geology, Denver, Colorado
June 26-July 2, 1961	7th Congress International Committee on Large Dams, Rome, Italy
June 18-21, 1962	4th National Congress of Applied Mechanics, University of California, Berkeley, California

Use of the Hydraulics Division Newsletter

You are urged to continue use of the Division Newsletter for announcements, inquiries, personnel news, committee reports, surveys and other items of interest to Division members. A short note summarizing the highlights of committee meetings is particularly requested. Suggestions for improvement of the Newsletter will be appreciated. All contributions are appreciated.

Deadline dates for Newsletter contributions: August 1960 issue—June 20; October 1960 issue—August 20.

James W. Ball, Editor
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